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Philadelphia Office

MEMORANDUM

American Chemical Services
Barrier Wall Degradation

B&V Project 46517.237
February 10, 1998

To: Steve Mrkvicka

From: Gary Snyder, Eric Lowry

This memorandum summarizes our cursory review of the life expectancy of the barrier wall, and the possibility of deterioration of the barrier wall due to the presence of organic hazardous waste and dewatering activities.

There are two parts to this barrier wall. First, the HDPE geomembrane, and second, the bentonite slurry surrounding the barrier wall. These are discussed separately in the following sections. Some additional discussion is also provided about constructability and operation issues.

HDPE Geomembrane

Two manufacturers of HDPE geomembranes, Polyflex and GSE, have developed guidance charts covering a wide range of chemicals and their impact on HDPE materials. Copies of these charts are attached for your reference. We have also included a memorandum regarding one of our projects which addresses this issue. References 1, 2, and 3 describe geomembrane compatibility and should be of use.

All of these documents recommend testing with site specific contaminants. Robert Koerner, an expert in the field of geosynthetics, concurs with the specific testing requirements, but also presents a concern for how individual chemicals mixed together in an uncontrolled manner (i.e. waste site) may react together and create different results than pure chemical tests done in a lab.

Typically, HDPE materials are relatively unaffected by contaminants and have a life expectancy of decades and probably longer. Reference 2 and conversations with Robert Koerner indicate that durability of geomembranes may be on the order of hundreds of years. However, detailed site specific data (i.e. compounds, concentrations, solubilities, diffusion coefficients, etc.) should be gathered to better evaluate compatibility. The ACS site has relatively high concentration of chlorinated solvents which have been proven to effect permeation or diffusion through HDPE. The presence of bentonite on either side of the HDPE barrier provides some redundancy and added protection against possible leakage.

Bentonite Slurry
There may be potential for chemical attack of the bentonite due to the contaminants in the groundwater and soil.

Natural bentonite combined with natural soil produce soil-bentonite backfill in conventional slurry walls which have demonstrated performance for decades (USEPA, 1997). Furthermore, it is widely believed among the industry that conventional slurry wall design life in excess of 100 years is a reasonable expectation in a favorable environment. However, the life expectancy of a bentonite barrier in contact with contamination is based on the type and concentration of chemical and the quantity of flow through the bentonite barrier. Bentonite's unique hydrating characteristics, and correspondingly low permeability, are impacted by ions in the pore water. Groundwater with naturally high concentrations or calcium, chlorides and some metals inhibit bentonite hydration and may increase barrier permeability. Contaminated groundwater with high ion concentrations of some metals or organics, as well as low pH, has negatively impacted the permeability of bentonite and other natural clays. Several references have been provided which discuss the issue of compatibility of contaminants and clays.

Compatibility between bentonite slurry materials and the site specific contaminants should have been determined as part of the ACS barrier design process. The pore volume exchange through the barrier should also have been determined as part of the design process. Results of this testing can provide more definitive information on the life expectancy of the site's bentonite barrier component.

Constructability and Operation Issues
The adequacy of the key between the bottom of the barrier wall and the confining strata underlying the site is critical to the performance of the containment (USEPA, 97). Minor imperfections in the key can allow significant leakage through the containment barrier. Ensuring that the key is constructed properly is an important aspect of barrier wall construction. Construction Quality Control and Quality Assurance (CQC/CQA) should verify that the key has been constructed according to plans to provide the necessary cut-off of groundwater flow under the barrier. Barriers constructed with conventional excavation equipment allow physical confirmation of the key elevation and key-in material by collecting samples during construction. Trenched systems, such as installed at ACS, do no allow the same level of inspection and may allow key material irregularities to go undetected, resulting in barrier leakage. Also, the confidence in composite barriers where geomembranes are installed in bentonite slurry may be limited by performance of the seams and assurance that the geomembrane was actually placed

uniformly and to the target depth within the trench. CQA/CQC of such installation can be problematic.

The groundwater hydraulic gradient affects flow through the barrier. This gradient is influenced by the natural hydraulic gradient and any groundwater pumping that may be performed. The larger the gradient, the greater the flow through the barrier. The life expectancy of the barrier components may be significantly influenced by the quantity of contaminated groundwater that moves through the barrier. Generally, active containment systems withdraw contaminated groundwater and cause inward flow of less contaminated groundwater. Obviously, barrier permeation with reduced contaminants results in less concern for chemical attack. However, certain site conditions can aggravate the barrier compatibility even during active pumping. Such has been the case with coastal bentonite based barriers which have been attacked by saline groundwater or possibly where the barrier changes external groundwater flow and exposes the barrier to differing contaminants.

Conclusions

The life expectancy of vertical barriers is not easily defined. Cited references suggest lives on the order of decades and even hundreds of years in moderately aggressive environments, assuming a quality constructed barrier. However, degradation mechanisms exist which negatively impact the barriers performance. In a recent barrier performance evaluation completed by USEPA, which we participated in, the following was determined regarding degradation measurement:

1. "The established industry baseline standard for post-construction degradation monitoring is that none is performed."
2. "Historical data that define the effects of long-term attack on vertical barriers are necessary to better understand the true functional life of such barriers."

Degradation mechanisms and constructability issues associated with composite barriers such as installed at ACS (i.e. key and continuity) raise concerns about use as a passive containment barrier, without interior pumping. Active containment with sufficient performance monitoring would be preferable, and should include performance monitoring of the necessary elements to detect any flow through imperfections or degradation from long term chemical attack.

Please contact either Gary Snyder at (215) 928-2233 or Eric Lowry at (215)928-2214 if you have any questions regarding the enclosed information.

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Geosynthetics '95

REF. 1

Effectiveness of Geomembranes as Barriers for Organic Compounds

J.K. Park
University of Wisconsin - Madison; USA

J.P. Sakti
Engineer; Indonesia

J.A. Hoopes
University of Wisconsin - Madison, USA

ABSTRACT

Double compartment tests were conducted to evaluate the transport of aqueous organic compounds through high density polyethylene (HDPE), very low density polyethylene (VLDPE), and polyvinyl chloride (PVC) geomembranes, which separated the two compartments. The concentration of methylene chloride (MC), toluene, trichloroethylene (TCE), and m-xylene was monitored in both upstream and downstream compartments over time. Organic compounds were detected in the downstream compartment in 20 to 200 hours for the 0.76, 1.52, and 2.54-mm thick HDPE geomembranes, in 8 hours for the 0.76-mm thick VLDPE, and in 9 hours for the 0.76-mm thick PVC. TCE had the greatest mass flux, followed by toluene, m-xylene and MC while m-xylene had the greatest partition coefficient, followed by toluene, TCE, and MC. A ten-fold increase in the initial aqueous concentration and a four-fold decrease in the geomembrane thickness increased the mass flux by 15 to 19 times. The mass flux increased by 45 to 97% when geomembranes were stretched in one direction by 5 to 8% of their original length.

INTRODUCTION

Composite double liner systems are required by the Environmental Protection Agency (EPA) for hazardous waste landfills and surface impoundments (EPA, 1988). Polyethylene (PE) geomembranes are the most common liners used for barriers or covers of hazardous chemicals in the environment, compared to polyvinyl chloride (PVC) and chlorinated polyethylene-chlorosulfonated polyethylene (CPE-CSPE), due to their excellent chemical resistance (Koerner, 1990). Because of their key role as barriers for isolating hazardous chemicals in landfills, it is important to thoroughly assess the effectiveness of geomembranes for organic compound containment.

Many organic compounds, which are hazardous to human health even at very low concentrations, have been found in landfills (Gibbons et al., 1992). Organic compounds are soluble in water to some degree. Thus, water will transport the dissolved phase of these substances as it percolates through a solid waste. This mixture is intercepted by clay liners and geomembranes. Organic compounds have been found to penetrate clay liners without significant retardation (Park et al., 1990).

Table 5 shows that the mass flux increased by 45 to 97% under tension while the partition coefficient increased by 11 to 93% under tension. Under tension, MC, which had the lowest mass flux, had the greatest increase in the mass flux, followed by m-xylene, toluene, and TCE. This indicates that slowly moving organic compounds may permeate at a greater rate under tension. The breakthrough time was faster when tension was imposed. The breakthrough times for 5 and 8% tension were practically the same. The breakthrough time decreased from 20 to 13 hours with a tension increase from 0 to 5% and 0 to 8%.

Table 5. Mass Fluxes and Partition Coefficients at Different Tensions with the Initial Aqueous Concentration of 100 mg/L and 0.76-mm Thick HDPE.

	0% Tension		5% Tension		8% Tension	
	F_{max} $mg/m^2 \cdot hr$	K	F_{max} $mg/m^2 \cdot hr$	K	F_{max} $mg/m^2 \cdot hr$	K
TCE	15.9	113	22.6	175	24.7	185
Toluene	14.6	140	22.4	190	21.9	205
m-Xylene	13.3	455	20.4	550	22.4	880
MC	2.9	6.3	4.8	7.2	5.7	7.0
T_b, hrs	20		13		13	

Effect of Geomembrane Type. Figure 7 shows normalized TCE concentrations versus time plots for 0.76-mm thick HDPE, VLDPE, and PVC at the initial aqueous concentration of 100 mg/L. PVC had the sharpest concentration decrease in the upstream compartment, followed by VLDPE and HDPE. HDPE had the highest equilibrium concentration and PVC had the lowest equilibrium concentration. Although PVC had the sharpest concentration decrease in the upstream compartment, the concentration increase in the downstream compartment was not as rapid as VLDPE.

PVC had much lower equilibrium concentrations than HDPE and VLDPE, implying that the partition coefficients of PVC are much higher than those of PE. As PVC in its pure state is more polar than the PE polymers, these results are expected for MC which has high solubility in water compared with m-xylene, toluene, and TCE which have very low solubility in water. The presence of crystallines in the polymer reduces the total uptake of an organic compound. In addition, the PVC geomembrane contains about 30% of plasticizers by weight. These plasticizers may have reduced the polarity of PVC and attracted more molecules of the less polar organic compounds (m-xylene, toluene, and TCE).

The mass fluxes and partition coefficients for 0.76-mm thick HDPE, VLDPE, and PVC at the initial aqueous concentration of 100 mg/L are summarized in Table 6 along with breakthrough times. VLDPE had the greatest mass flux, followed by PVC and HDPE for non-polar organic compounds while PVC had the greatest mass flux, followed by VLDPE and HDPE for the polar compound, MC. VLDPE had approximately 1.8 to 3.2 times greater partition coefficients and PVC had 6.2 to 8.3 times greater partition coefficient than HDPE, depending on the organic compound. PVC appeared to contain a significant amount of plasticizers, which resulted in a rather higher partition coefficient even for non-polar compounds. For each organic compound with the same geomembrane thickness, VLDPE had breakthrough times more than 2.5 times as fast as HDPE, whereas PVC had almost the same breakthrough time as VLDPE. Hence, organic compounds appear to diffuse through VLDPE and PVC much fast than HDPE.

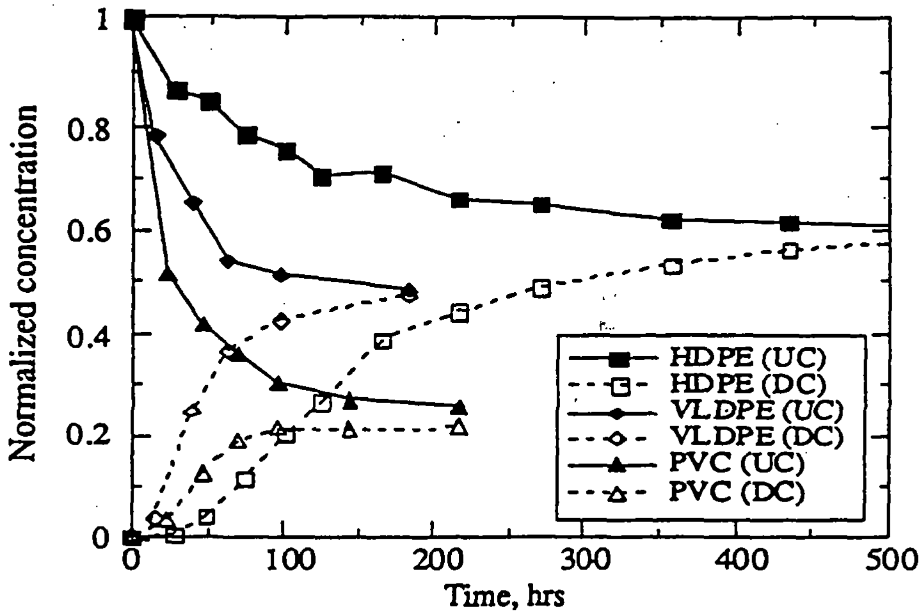

Figure 7. Normalized TCE Concentration Changes at Upstream and Downstream Compartments for 0.76-mm Thick HDPE, VLDPE, and PVC at the Initial Aqueous Concentration of 100 mg/L.

Table 6. Mass Fluxes and Partition Coefficients at Three Different 0.76-mm Thick Geomembrane Types with the Aqueous Initial Concentration of 100 mg/L.

	HDPE		VLDPE		PVC	
	F_{max} $mg/m^2 \cdot hr$	K	F_{max} $mg/m^2 \cdot hr$	K	F_{max} $mg/m^2 \cdot hr$	K
TCE	15.9	113	43.4	218	20.6	770
Toluene	14.6	140	40.3	245	13.4	1160
m-Xylene	13.3	455	23.6	800	5.0	3300
MC	2.9	6.3	10.2	20	22.2	39
T_b, hrs	20		8		10	

SUMMARY AND CONCLUSIONS

The confined double compartment test allowed monitoring of concentration changes in both upstream and downstream compartments with or without tension at various initial aqueous concentrations and geomembrane thicknesses. From a series of the confined double compartment tests, the following conclusions can be drawn:

(1) Methylene chloride, toluene, trichloroethylene, and m-xylene were detected in the downstream compartment in 20 to 200 hours for the 0.76, 1.52, to 2.54-mm thick HDPE geomembranes, in 8 hours for the 0.76-mm thick VLDPE, and in 9 hours for the 0.76-mm thick PVC at the initial aqueous concentration of 100 mg/L

(2) The breakthrough times for 5 and 8% stretched HDPE geomembranes were approximately the same but about 48% faster than the unstretched geomembrane. The breakthrough

times through VLDPE and PVC were almost the same but more than two times faster than HDPE.

(3) The partition coefficient increased when the HDPE geomembrane was stretched from 0 to 5% but remained constant when the geomembrane was stretched from 5 to 8%. PVC had much higher partition coefficients than VLDPE, while HDPE had significantly lower partition coefficients than VLDPE.

(4) The mass flux was significantly affected by the initial aqueous concentration, thickness, tension, and type of geomembrane. The mass flux increased from 1 to 15.9 $mg/m^2 \cdot hr$ for TCE when the initial aqueous concentration increased from 10 to 100 mg/L with 0.76-mm thick HDPE. The mass flux decreased by a factor of 7 to 22 depending on the organic compounds when the thickness increased from 0.76 to 2.54 mm. The mass flux increased by 50 to 97% under tension. The mass flux of MC in PVC was significantly greater than that in VLDPE and HDPE while the mass flux of non-polar compounds in PVC was 2.1 to 4.7 times lower than VLDPE.

(5) The mass flux by permeation was estimated to be more than two orders of magnitude greater than the mass flux through holes in the geomembrane.

(6) There appear to be two phenomena which control mass transfer in the geomembranes: partition; and diffusion. Methylene chloride had the lowest partition coefficient, followed by trichloroethylene, toluene, and m-xylene. The partition coefficient for HDPE geomembrane appeared to be almost constant at aqueous concentrations less than 100 mg/L.

(7) The time of permeation increased approximately in proportion to the square of the geomembrane thickness. It would require a thickness of 8 cm for no organic compound permeation in 25 years if organic compounds exist in the leachate for 25 years at the same concentration.

ACKNOWLEDGMENTS

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REF. 2

Post-it® Fax Note 7671	Date 27 Jan 98	# of pages ▶ 3
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The durability of HDPE geomembranes

By L.G. Tisinger and J.P. Giroud

Excellent papers have been written on the durability of high density polyethylene (HDPE) geomembranes. Since the subject is very complex, however, many of these papers can be understood only by polymer scientists. Because information on the durability of HDPE geomembranes is very important, such information needs to be presented to the wide range of geomembrane users.

In this article, aspects of materials' durability that relate to the composition and/or structure of the material used in the geomembrane will be discussed. Mechanical actions, including stress cracking, and aspects related to the durability of the geomembrane seams will not be addressed.

From low to high density

Polyethylene is a polymer. A polymer is a molecule that has many units (from the Greek, poly, which means many, and meros, which means part). In contrast, a monomer is a single unit (from the Greek monos, which means single). Polymers are made from monomers through a reaction called polymerization.

For example, a polyethylene polymer results from the polymerization reaction of the ethylene monomer (Seymour and Carraher, 1981).

Production of polyethylene began in the mid-1930s from a process using high pressure and high temperature (Brydson, 1982). In the mid 1950s, new reaction conditions were introduced in which polyethylene was produced at lower pressures and lower temperatures than before.

As a result, a new variety of polyethylene was made that had a higher softening point, a higher density and more rigidity than earlier types.

This new variety of polyethylene was appropriately named high density polyethylene, while the name low density polyethylene (LDPE) became used to designate the type of polyethylene produced with the early process.

Anatomy of HDPE

The high density of HDPE results from the presence of many crystals of polyethylene molecules within its structure. Crystals are regions in which matter is ordered and densely packed.

The crystalline regions are connected by less organized, or amorphous regions, hence the terminology semicrystalline structure. The amount of crystalline regions in a material is typically expressed as crystallinity, a ratio that varies between 0 percent for a totally amorphous material and 100 percent for a totally crystalline material. Crystallinity, measured by differential scanning calimetry, is the ratio of the energy required to melt a given HDPE to the energy required to melt a totally crystalline HDPE.

Because they are composed of densely packed matter, crystals are essentially impermeable to liquids and chemicals. Clearly, a relationship exists between the number of crystals, the density of polyethylene and the impermeability of the geomembrane.

HDPE used to produce geomembranes is made not only from ethylene. It also contains some comonomer (a monomer in addition to ethylene at a proportion of approximately 1 percent to 3 percent), such as butene, hexene or octene. Comonomers result in more branching on the polyethylene molecules of HDPE, which usually improves HDPE materials' flexibility and environmental stress cracking resistance (Bourgeois and Blackett, 1990).

As more branching slightly increases the distance between parallel long-chain molecules, however, it increases HDPE material permeability and reduces its chemical resistance, but by amounts that are generally considered insignificant.

HDPE geomembranes are not made from HDPE only. They also contain additives, such as carbon black and antioxidants. The resulting material is called the HDPE compound and it contains approximately 97 percent HDPE, 2.5 percent carbon black, and 0.5 percent antioxidants. Note that HDPE geomembranes do not contain plasticizers.

Chemical reactions

HDPE is chemically resistant for two reasons. First, as all members of the polyethylene family, HDPE is essentially inert. Second, as discussed earlier, because of its high density, HDPE has a low permeability; therefore, it resists penetration by chemicals. Under certain conditions, however, HDPE can react with chemicals. A chemical reaction between a material and a chemical occurs when the chemical modifies the structure of the molecules making up the material.

Reaction of HDPE with chemicals is generally limited to oxidizing agents, such as nitric acid and oxygen. In other words, oxidation is the predominant mechanism of chemical reaction of HDPE. Oxidation is a step-wise process.

The polymer first absorbs energy, provided by heat, UV radiation and/or high-energy radiation (radioactivity). This absorption excites the polymer molecules, causing them to break, forming highly reactive fragments referred to as radicals. This mechanism is called chain scission. The radicals then react with oxygen, forming even more radicals.

As the process proceeds, an increasing number of radicals are formed. The process is terminated only when the radicals either react with antioxidants or recombine, or when energy is no longer supplied (Brydson, 1982; Rodriguez, 1970; and Seymour and Carraher, 1981). If oxidation occurs, it causes the molecular weight of molecules to decrease, making the HDPE material soften and embrittle, thereby becoming subject to stress cracking. Oxidation occurs only if two conditions are present.

The first condition is a high concentration of the oxidizing agent. The second condition is that the material must receive a sufficient supply of energy to activate the reaction.

When the conditions are not present—which is often the case—HDPE is not attacked. This is confirmed by reported cases of EPA 9090 tests conducted to evaluate the chemical compatibility between HDPE geomembranes and municipal waste or hazardous waste leachates from modern waste disposal facilities, which indicate no detectable deterioration of the properties of HDPE geomembranes (Ojeshina et al., 1984; and Dudzik and Tisinger, 1990).

Physical interaction

Another potential mechanism of HDPE degradation is physical interaction. Physical interaction of HDPE with a chemical occurs when HDPE, without experiencing change in the structure of its molecules, absorbs the chemical, usually organic. Organic chemicals can interact with HDPE, because like HDPE, they are nonpolar, and therefore, have similar intermolecular forces (cohesive forces) holding adjacent molecules together. The most typical mechanism of physical interaction involving HDPE is solvation.

Solvation Solvation is a physical process by which solvent molecules are absorbed into a material. Solvation causes a polymeric material to swell (which increases its permeability) and to soften, a process often referred to as plasticization. A limited degree of swelling and softening is, to some extent, reversible: The geomembrane more or less retrieves its original dimensions and properties if the solvent is removed by evaporation. The ultimate degree of solvation is dissolution, where the molecules of the initially solid material are dispersed in the solvent. Of course, this mechanism is not reversible.

Typical solvents that may cause solvation of HDPE are aromatic solvents, such as benzene, toluene, xylene and halogenated solvents, such as chloroform, methylene chloride and trichloroethylene. These solvents cause some degree of solvation of HDPE at ordinary temperature. Dissolution of HDPE by these solvents, however, will not occur at ambient temperature.

A USEPA ad hoc committee has concluded that polymeric landfill lining materials should maintain their integrity in waste diposal environments in "terms of hundreds of years."

In fact, no known solvents can dissolve HDPE at room temperature. Typical waste disposal facility temperatures should not exceed 50 C, which is significantly below 80 C, the temperature at which some solvents may begin to dissolve HDPE. These solvents should, therefore, not cause complete dissolution of HDPE geomembranes under waste disposal facility conditions.

Moreover, the solvents must be present at very high concentration to affect HDPE, a condition that is not observed in waste disposal facilities.

Extraction Extraction is a mechanism of physical interaction between polymeric compounds and chemicals. It is a process by which chemicals and heat cause additives, such as plasticizers and antioxidants, to leach out of the polymeric compounds.

HDPE compounds used to produce geomembranes do not contain plasticizers; however, their antioxidants can be extracted. Such an extraction typically requires a very high concentration of chemical, a condition typically not present in a waste disposal facility. Moreover, most modern antioxidants have a high molecular weight and are physically entangled among the polyethylene molecules. Such physical entanglement greatly reduces the ability of chemicals to extract antioxidants. As a result, HDPE geomembranes do not undergo significant loss of antioxidants by extraction.

Energy and environment

In all the potential mechanisms of degradation described above, energy plays a crucial role. In geomembrane applications, the most typical sources of energy are heat and ultraviolet (UV) radiation; both conditions often occur through direct exposure to sunlight. Also, exposure to high-energy radiation (radioactivity) can induce reaction of HDPE with oxidizing agents. High-energy radiation also may cause HDPE to crosslink, that is, to form chemical bonds between adjacent polyethylene molecules. As a result, HDPE may harden and become brittle. Again, for this to happen, HDPE would have to be exposed to large doses of high-energy radiation (Whyatt and Farnsworth, 1990).

In the absence of either oxygen or energy, oxidation, the predominant mechanism of chemical reaction of HDPE, cannot occur. Typical waste disposal facility environments are anaerobic, eliminating the possibility for oxidative degradation of HDPE geomembranes once they are buried (Haxo and Haxo, 1989).

In addition, the supply of energy is limited, because there is no light and because geomembranes are usually protected by a layer of soil, which insulates them from heat generated by decomposition of waste.

Some oxidation of HDPE geomembranes can occur as the result of their exposure to sun during installation. Such oxidation is limited and superficial, however, because carbon black, which is an additive used in most HDPE geomembranes, absorbs sunlight, preventing it from penetrating the geomembrane (Whitney, 1988).

Furthermore, the effects of oxidation should be limited, because HDPE geomembranes contain antioxidants, additives that stabilize radicals generated by HDPE's absorption of energy. Information on the durability of HDPE geomembranes that are permanently exposed can be obtained from experience gained in observing the performance of existing facilities.

If not attacked, could HDPE simply age?

Aging refers to changes that occur in materials when they are subjected to the type of temperate conditions in which a human could survive (but would age)—no contact with liquid chemicals, moderate ambient temperature, no exposure to UV radiation or radioactivity, no supply of oxygen beyond that naturally present in air, etc. Studies have indicated that the effect of such conditions on HDPE materials is very slow.

For example, test results obtained from polyethylene films stored in a ventilated box exposed to desert, temperate and tropical environments for 15 years, have shown negligible changes in crystallinity and minimal evidence of oxidation (Moakes, 1976).

Resistance to aging is best evaluated by observations of actual performance in service. Polyethylene has a long track record of successful uses. Polyethylene was first synthesized in 1933, and became commercially available in 1937.

The use of polyethylene for cable sheathing began in 1942 (Gilroy, 1985). Since then, polyethylene has been the material of choice for the protection of transatlantic cables.

The first HDPE geomembranes were used in 1973 in Europe (Knipschild, 1984) and in 1974 in the United States. To date, HDPE geomembranes have been used, exposed or buried, for 20 years. Wherever they have been properly protected against mechanical failures (including stress cracking), HDPE geomembranes have performed satisfactorily. The performance of HDPE geomembranes for 20 years confirms the successful performance of HDPE in other outdoor applications, such as cable sheathing and buried pipes, for more than 40 years.

How long will geomembranes last?

A question frequently asked about geosynthetics and geomembranes in particular is, "How long will they last?" To answer this question, some clear conclusions can be drawn from the facts presented earlier.

Experience has shown that exposed HDPE materials, including geomembranes, can perform satisfactorily for decades if they are protected from mechanical aggressions.

In waste disposal facility environments, once HDPE geomembranes are buried, only little energy should be acting on them, and in addition, the supply of oxygen should most likely be very low. In the absence of an aggressive environment, therefore, HDPE geomembranes should last for a very long time in waste disposal facilities.

A U. S. Environmental Protection Agency (USEPA) ad hoc committee on the durability of polymeric landfill lining materials has concluded that the polymeric landfill lining materials should maintain their integrity in waste disposal facility environments in "terms of hundreds of years" (Haxo and Haxo 1988). This conclusion is consistent with durability evaluations made using the Arrhenius model (Koerner et al., 1990). One can conclude, then, that in properly designed and constructed facilities, HDPE geomembranes should be able to protect ground water from leachate for hundreds of years, which is long after leachate generation has stopped.

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REF. 3

vs. Containment of Solid/ Liquid Waste Sites.

Robert M. Koerner, GRI, Drexel University, Philadelphia PA, USA.

Professor Robert Koerner certainly needs no introduction to any person familiar with geosynthetics. As Director of the Geosynthetics Research Institute he is at the leading edge of knowledge.

We sincerely thank him for making this feature length article available for publication and note that it replaces his usual column. **GW.**

Cover picture - left. Dura Avenue Landfill Leachate Collection System, Toledo, Ohio, USA. Four feet (ca. 1.2m) wide HDPE panels driven to a depth of more than 30 feet (more than ca. 9m). The project was unique as the panels were installed some 12 feet (ca. 3.65m) out into a frozen river. A fuller report can be found on page 14. [Credit: GSE Lining Technology, Inc., Houston, Texas, USA].

Cover picture - right. Former Schooteroog Landfill, Haarlem, The Netherlands. HDPE panels, 2.5m (ca. 8.2 feet) wide and 2mm (ca. 0.08 inches) thick, were installed in a 150mm (ca. 6 inches) thick bentonite/cement slurry wall. The depths of installation were 12m - 15m (ca. 39 - 49 feet). A fuller report can be found on page 16. [Credit: Geotechnics Holland B.V., Amsterdam, The Netherlands].

The size and scope of remediating waste sites containing various solid and liquid materials, e.g., abandoned landfills and surface impoundments, is simply awesome. The size of these sites range from the radioactive waste at Chernoble in the Ukraine to a small construction debris site in your local neighborhood -- and the risk is every bit a variable as the size. Arguably, the sites fall into three general categories.

a) Federally-owned sites which are usually owned and/or operated by the military or the power related segment of the government. Such sites in the USA are often operated by the Department of Energy and the Department of Defence.

b) Privately-owned sites which are often owned/operated by a wide range of industrial corporations which have numerous facilities containing solid/liquid waste materials.

c) "Nobody"-owned sites which are abandoned and appear randomly across every country and indeed around the world. In the USA, these sites are the focus of CERCLA regulations and are referred to as Superfund sites.

The U. S. Environmental Protection Agency (and sister agencies in countries around the world) make regular attempts at assessing the magnitude of the situation. Unfortunately, the resulting number of sites and quantities of waste involved continue to grow as the various investigations are undertaken. By anyone's standard, the number of sites and quantities involved are enormous and action is absolutely necessary. Whatever action is taken it can fall under one of two classifications: either "remediation" or "containment".

Some of the techniques considered as remediation are as follows (in no particular order):

° soil/waste washing
° waste solidification
° waste vitrification
° complete incineration

With all of the above, serious consideration must be given to a number of issues; for example, the targeted degree of remediation, i.e., "how clean is clean?", the unknown impacts of aggravated water and air pollution, a number of safety issues to the local community, safety issues to the workers performing the cleanup, obtaining permits and approval to commission the remediation work, and a host of technical/societal/political issues. Finally, someone must consider the issue of cost and how (i.e., "by whom") it will be paid.

It has become apparent to many, that the cost of remediation is dwarfing the capability of private and public funds to accomplish the task at hand. In the extremes, the cleanup of Chernoble will probably never be within the economic grasp of the fledging Ukrainian state for remediation. It is almost as unlikely to expect a local neighborhood community to clean up the adjacent landfill to "five-nines", i.e., 99.999%, of the perfect environment. The remediation of existing contaminated sites has been extremely slow over the past 15 years. The funds expended in the years of Superfund vis-a-vis the number (or cubic meters) of cleaned sites speaks for itself.

In the writer's opinion, it is high time for a paradigm shift from the remediation of contaminated sites, to the containment of them.

The shift to a containment strategy has often been voiced, but the inherent aspect of leaving our wastes for future generations is admittedly unsavory. While the author agrees, to do nothing with these sites is even more disturbing. And the latter is the current situation! Action is necessary now, containment is the strategy, costly than any remediation option. How much lower of a cost depends on the particular site and its waste contents. One known project puts containment at 1/100-th of the cost of remediation. It is unknown if this is typical.

Regarding the containment of waste sites, three elements are generally involved; cover, walls and floor. Considering the covers of solid waste sites, the state of-the-practice is well advanced. The individual components are clearly defined (with geosynthetics playing a key role) and the open literature is abundant in this regard. Considering the walls of solid waste sites, the state-of-the-practice is also well advanced. A main issue, however, is the necessity of using a geomembrane in a backfilled trench. Also its depth is a contentious issue, i.e., the choice between keying into an acquitard or using a "hanging wall" design. Considering the floors of solid and liquid waste sites, the situation is quite unsettled. Techniques of drilling through the waste then jet grouting, directional drilling and grouting, or tunnelling beneath the waste have all been proposed, but much more remains before reliable methods are available. Nevertheless, containment via covers and walls are usually adequate for most sites and are well within our grasp.

Two remaining issues need commentary; performance monitoring and system permeance. On performance monitoring downstream wells could be relied upon, but geosynthetics offer a better and less expensive long-term alternate. Double barrier containment with intermediate leak detection is the key. As with liners beneath landfills, two geomembranes with a geonet as an intermediate drainage layer can be used in both the covers and the walls of sites to be contained. In this regard, 100% leak detection coverage is available for the length of time that the materials are intact. This brings up the second issue of system permeance. The geosynthetics of today have excellent longevity when backfilled in a timely manner. Recent estimates of 1000-year lifetime for high density polyethylene (HDPE) are not beyond reach. For example, the time for depletion of antioxidants (i.e., with no polymer degradation of the polymer whatsoever) is between 50 and 200 years, depending on the local conditions of the geomembrane. Indeed, the time to subsequent half-life of the engineering properties of a properly formulated HDPE geomembrane is many centuries and eminently suited for the containment of waste sites.

In summary we must begin the paradigm shift from remediation to containment. While containment as the ultimate "fix" is not completely fulfilling, it is the only practical solution available. The procedures, designs, contractors, materials, etc., are fully available and the situation is quite economical. Without emphasising the obvious, geosynthetics play a key role in any waste containment strategy. They are well positioned to do so.

Figure 1 a. Geomembrane in a vertical wall seamed to a geomembrane in the cover.

Figure 1 b. Horizontal or vertical overlap of a geomembrane in the cover.

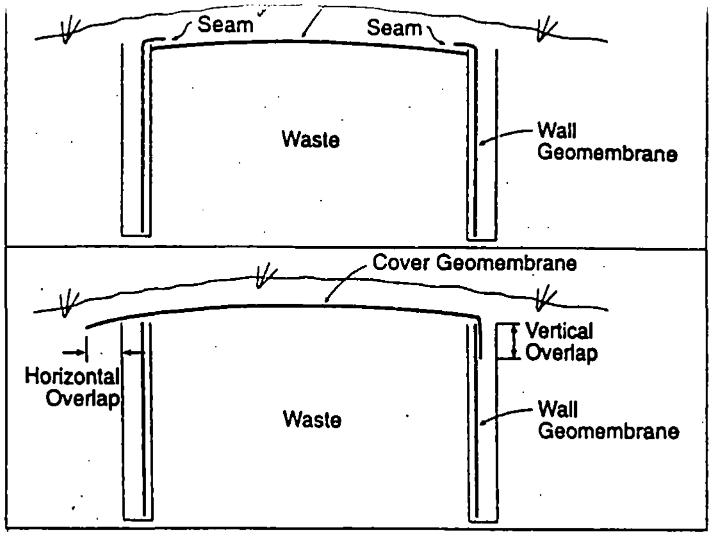

Figure 1 a & b above. Termination of the top of geomembrane vertical barriers. (Credit: GRI, Drexel University, Philadelphia PA, U S A).

Figure 2 a. Keyed (and grouted) into the aquitard.

Figure 2 b. Deep (hanging) wall beneath the waste.

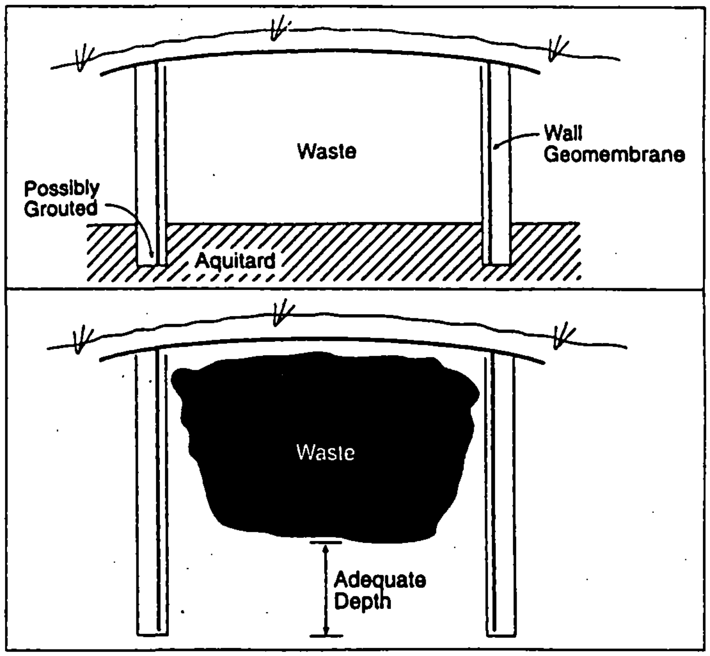

Figure 2 a & b above. Termination of the toe (bottom) of the geomembrane vertical barriers. (Credit: GRI, Drexel University, Philadelphia PA, U S A).

Robert M. Koerner, Director,
GRI, Drexel University,
33rd. & Lancaster Walk,
Rush Building - West Wing,
Philadelphia, PA 19104, USA.
Tel. + 1-215-895-233.
Fax. + 1-215-895-1437.

Editor's note:

There will be 2 meetings at EuroGeo 1 to discuss the GRI and it's opening up into a broader based Geosynthetics Institute:

GRI/GSI Open Meeting.

2 October, 14.00 to 15.30, in Room 2.14 - Amazon, The Promenade.

GRI/GSI Members-Only Meeting.

2 October, 16.00 to 17.30, in Room 2.14 - Amazon, The Promenade.

Contact address on the left. GW

EVALUATION OF SUBSURFACE ENGINEERED BARRIERS AT WASTE SITES

Volume I

Prepared for:

U.S. Environmental Protection Agency
Office of Solid Waste and Emergency Response
Office of Emergency and Remedial Response
Washington, DC 20460

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Date Prepared:	August 30, 1996
Contract Number:	68-W5-0055
EPA Work Assignment Manager:	Amy Mills
Telephone:	(202) 260-0569
Prepared by:	PRC Environmental Management, Inc.
PRC Work Assignment Manager:	Paul Dean
Telephone:	(703) 287-8806

In general, three trench slurry compatibility tests should be conducted. Conduct of more than three tests is better than acceptable, and if fewer than three, less than acceptable.

The compatibility of trench slurry was evaluated at most of the sites studied; the number of tests varied from 2 to 5.

Testing of Backfill Permeability

The permeability of the backfill used to construct the barrier wall is a key design parameter that should be tested adequately. For the soil-bentonite technique, the objective is to establish proportions of on-site or imported materials needed to achieve the target permeability and physical properties of the barrier backfill. References and sources differed significantly on what constitutes standard practice. Site conditions, availability of borrow materials, and procedures for testing permeant compatibility affect the number of tests required. However, the consensus average was approximately three permeability tests of the backfill (the same or similar batches), using acceptable laboratory procedures that simulate in situ conditions. Conduct of three tests is acceptable. Conduct of more than three tests is better than acceptable, and of fewer than three, less than acceptable.

The permeability of backfill at the sites studied varied from 1 x 10-6 to 9 x 10-9 cm/sec. The number of tests conducted to verify the permeability varied from 2 to 5.

[redacted]

Since chemical reaction with contaminants can increase the permeability of the backfill, the long-term compatibility of backfill with the in situ soils and groundwater should be analyzed. If contaminant reaction to the backfill is unknown, more tests are required; if the contaminant reaction is known, fewer tests are required. Typically, several permeability tests of multiple pore volumes are performed to simulate a long-term condition and identify degradation through changes in permeability with time. Such tests often are combined with the testing of permeability of the backfill. Conduct of 3 tests is acceptable. Conduct of more than 3 tests is better than acceptable, and of fewer than three, less than acceptable.

The compatibility testing was done at all sites at which leachate or contaminants were encountered. The extent of testing varied from site to site, with rigorous testing done at some sites and very limited testing at other sites.

Barrier Penetration

Subsurface utilities present along the barrier wall alignment and located below the water table must be delineated, rerouted, or protected with watertight connections. If such conditions were not considered, the site was rated less than acceptable; if the contractor designed solutions during construction, it was rated acceptable; and if the engineer investigated the problems and designed solutions during design, it was rated better than acceptable. Barrier penetrations were encountered at only a few of the sites studied. In all those cases, the barrier penetrations were investigated and accounted for in the design by the engineer.

Surface Cap

The surface or wall cap over the barrier wall alignment must protect against erosion, desiccation, and long-term physical disturbance of the barrier. Since the earthen barrier materials are primarily clays and bentonite, they are susceptible to desiccation that leads to the development of macropores and secondary permeability in the upper section of the barrier. If the barrier wall is protected from desiccation with less than 1 foot of cover soil, the wall is rated less than acceptable; if the wall is protected with 1 to 2 feet of clay cap, the wall is rated acceptable. If the wall is protected with more than 2 feet of clay cap placed in a controlled manner, the wall is rated better than acceptable. If no cap is provided, the site is considered less than acceptable; if physical protection is provided, it is considered acceptable; if a permanent structural cap is provided, the site is considered better than acceptable.

At all the sites studied, a surface cap had been provided over the barrier wall alignment.

Interface of Barrier and Cap

The cap and barrier wall form an integrated containment system that minimizes entry of water into the waste area or its migration out of the area. If no surface cap is provided, the site is rated

[redacted]

Monitoring environmental degradation of vertical barriers after construction is not practiced widely. Detection of degradation processes would allow the introduction of corrective measures or perhaps lead to preventive design modifications. Degradation mechanisms can include chemical attack (for example, a high concentration of chlorinated solvents), inhibited bentonite hydration caused by saline or hard water, desiccation of earthen barriers in a cyclic vadose zone, and corrosion of metal-sheeted structures. The established industry baseline standard for postconstruction degradation monitoring is that none is performed. Testing for degradation would involve some form of direct monitoring.

Often during the design phase, chemical compatibility testing is performed if there is concern about chemical attack on the vertical barrier, especially in the case of earthen barriers. This laboratory testing typically involves permeating backfill samples with 3 to 5 pore volumes of contaminated permeant. Such tests may not simulate adequately in-situ, long-term conditions at the barrier. For approximately half the barriers studied some compatibility test was performed in the design phase (see Section 3.2). However, postconstruction analysis of chemical breakthrough and degradation was reported for only 2 of the 36 sites studied. At no site were periodic long-term degradation monitoring data collected.

For nonearthen barriers, particularly geomembrane and sheeting, are monitored for degradation differently than are earthen barriers. At 1 site having steel sheeting containment, conventional ultrasonic testing was performed after years of operation to determine corrosion. No decreased performance was noted. Vinyl or plastic sheeting offers obvious corrosion advantages over steel. Geomembrane vertical barriers have benefited by research on landfill liners including extensive laboratory simulations and field efforts that involve exhuming installed geosynthetics. Such tests have indicated lives of hundreds of years for geomembranes buried underground and not subject to degradation caused by ultra violet (UV) light.

Historical data that define the effects of long-term attack on vertical barriers are necessary to better understand the true functional life of such barriers.

3.4.1.2 Range of Findings

BARRIER CONTAINMENT TECHNOLOGIES FOR ENVIRONMENTAL REMEDIATION APPLICATIONS

(A project sponsored by E.I. DuPont de Nemours & Company)

Edited by

Ralph R. Rumer and Michael E. Ryan

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results of a theoretical and field investigation of the coupled hydromechanical behavior of fractured rocks. Their results validate the soundness of the conceptualized constitutive relations governing the coupled hydromechanical behavior of rock masses.

3.5 TEST METHODS AND COMPATIBILITY

3.5.1 Test Methods

It is not within the scope of this book to provide a comprehensive summary of literature relating to test methods for hydraulic conductivity. The reader is referred to other sources for a more complete treatment of the subject (Bowders et al., 1986; Daniel et al., 1984; Dunn and Mitchell, 1984; Lambe and Whitman, 1979; Olson and Daniel, 1981).

Studies have shown that the type of permeameter has little effect on the hydraulic conductivity of a compacted clay when measured in a laboratory using water as the permeant (Daniel et al., 1985); especially when the impact of effective stress is considered (Manuel et al., 1987). Sidewall leakage, and accompanying sample shrinkage, contributes to the magnitude of the hydraulic conductivity increases in studies using fixed wall permeameters with concentrated organic permeants. Regardless of the type of permeameter used, the ability to maintain constant influent chemistry throughout the test is necessary (Evans and Manuel, 1985). There are a number of testing parameters that can affect the outcome of a laboratory compatibility test (Evans and Fang, 1983, 1988; Zimmie et al., 1981).

Nordquist et al. (1986) describe the results of measuring the hydraulic conductivity of clay liners both in the field and in the laboratory. Averages of field and laboratory results were within one order of magnitude and indicate the need for careful field quality control methods. Grube (1990) reviewed the different problems associated with measuring the performance of clay containment barriers. An evaluation of the hydraulic performance of barrier dikes and cutoff walls is more complex than in analyzing the behavior of normal undisturbed soil systems. Anderson et al. (1991) described a calibration chamber suitable for the preparation of uniform clay beds in which the performance of full-size field test devices may be studied. McBride and Baumgartner (1992) described an inexpensive slurry consolidometer that uses porous polyethylene as the permeable barrier for sample dewatering and permits monitoring of sample pore-water pressures less than 100 kPa with a portable pressure transducer. Tan et al. (1992) described a practical method of measuring the *in situ* slurry density profile with depth using a submersible gamma source, backscatter-type nuclear density gauge.

It is also difficult to clearly establish the compatibility test duration. Although passing of two or three pore volumes has been the generally accepted practice, it is not clear that this criterion is always suitable. Studies of clays

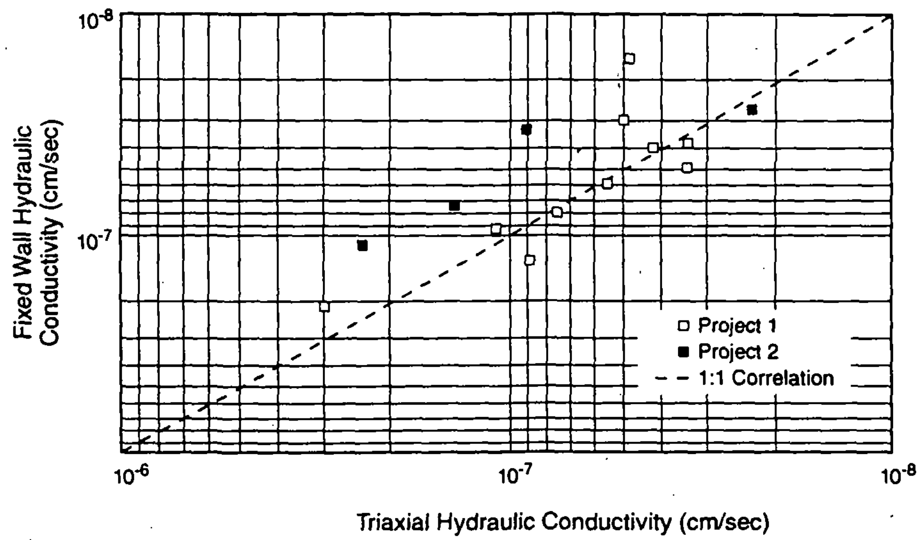

Figure 3.22 Permeameter test results (data from Day, 1992; Barvenik, 1992).

permeated with concentrated organics have generally shown dramatic hydraulic conductivity increases within two pore volumes. However, where dissolutioning of the soil structure is occurring, the time required to establish a new equilibrium hydraulic conductivity may be much greater than the time required to pass two pore volumes. Some data have shown that the number of pore volumes may be less important than the time of exposure (Bodocsi et al., 1987). Pierce and Witter (1986) suggest one pore volume of fluid with the condition that the slope of hydraulic conductivity versus pore volume displacement be essentially zero. Alternatively, Bowders (1988) suggests two pore volumes of flow and that the influent chemical concentrations be essentially the same as the effluent chemical concentrations.

It has been shown that hydraulic conductivity testing can be accomplished relatively rapidly on-site using a rigid wall permeameter to achieve results comparable to triaxial tests (GKN, 1989b). Typical data are shown in Fig. 3.22. The determination of hydraulic conductivity must be at the same gradient and consolidation pressure as expected in the field since soil–bentonite backfill hydraulic conductivity is stress dependent (McCandless and Bodocsi, 1988). In general, rapid field permeability tests are conducted in an American Petroleum Institute (API) filter press rigid wall permeameter.

[redacted]

A major concern in the application of vertical cutoff walls to site remediation is the compatibility of the barrier material with the site specific contaminants. This section presents results reported in the literature and provides assessment of the findings with respect to the compatibility of vertical barrier wall materials. Readily available information regarding clay mineralogy and clay–water–electrolyte systems is not presented here. The reader is referred to existing publications (Grim, 1968; Mitchell, 1993; Van Olphen, 1977) for detailed information.

Widespread concerns regarding the potential incompatibility of clayey materials with subsurface contaminants began to surface in the late 1970s and early 1980s. In a paper describing slurry cutoff walls for control of hazardous wastes, Ryan (1985) does not present any compatibility data and states that "We have yet to find a leachate whose effect on the soil–bentonite backfill cannot be counteracted by relatively minor changes in the constituents." In a paper describing the use of slurry walls, D'Appolonia (1980) presented data indicating that certain inorganic chemicals could cause increases in the hydraulic conductivity of the filter cake. Early work, published while studies were still in progress, indicated the potential for large increases in hydraulic conductivity as a result of permeation of clayey soils with concentrated organic liquids. As a result of this concern, a number of investigations were undertaken to further investigate the phenomenon. However, most of the studies dealt with compacted clays, which are considerably denser and have much lower water contents than slurry walls.

Brown and Anderson (1983) found that permeation of compacted clays with concentrated organic fluids can lead to increases in hydraulic conductivity from one to three orders of magnitude. They assessed their findings in light of the accepted colloidal models of soil behavior and concluded that the behavior was consistent with these models (such as Gouy–Chapman). The studies were performed using rigid wall, compaction mold permeameters. Anderson et al. (1985) found similar dramatic increases in hydraulic conductivity when testing bentonite slurries with concentrated organics (xylene and methanol) in double ring fixed wall permeameters.

Acar et al. (1985) also found increases in the hydraulic conductivity of compacted kaolinite when exposed to concentrated organics. Importantly, these studies examined the influence of confining pressure and fluid concentration on changes in hydraulic conductivity. For permeation with concentrated organic fluids, dramatic increases in hydraulic conductivity were observed for tests done in rigid wall permeameters but, as the confining pressure increased in tests using flexible wall permeameters (triaxial cells), the magnitude of the hydraulic conductivity increases decreased. Stated another way, the ratio between the final and initial hydraulic conductivity decreased as the confining pressure increased. The study also found that all tests conducted using organics at low concentrations resulted in slight decreases in hydraulic conductivity. The hydraulic conductivity data, as well as indicator data (Atterberg limits and free swell) were found to be consistent with changes expected from variations in surface interaction forces between the colloidal clay particles.

In another study, it was found that the hydraulic conductivity ratio (the ratio between the hydraulic conductivity with the contaminant to that with water) increased with decreasing activity, whereas, volume decreases were greater with increasing activity (Acar and D'Hollosy, 1987). Activity is defined as the plasticity index normalized with respect to the clay fraction; high activity clays, such as bentonite, are thus subject to greater ranges of shrinking and swelling than low activity clays, such as kaolinite. The results indicate that while soil fabric changes are important in controlling hydraulic conductivity, at increasing activity the volume change becomes the controlling factor.

Studies of soil–bentonite materials in triaxial permeameters demonstrated significant increases in hydraulic conductivity with concentrated organic fluids, whereas, virtually no effect was observed for the same fluids dissolved in water at low (up to 30,000 ppm) concentrations (Evans et al., 1985b and c).

The effect of inorganic permeants on bentonite was examined by Alther et al. (1985) employing filter press and cracking pattern tests. These investigators found that there was an increase in hydraulic conductivity with increasing electrolyte concentration. They also found that divalent cations had a greater impact than monovalent cations. These data were generally consistent with that expected from an examination of the colloidal behavior as described by the Gouy–Chapman model. When all else is held constant, the permeability should also increase with smaller hydrated ions. The data were inconclusive in this regard.

In another study of inorganic permeants, the pH of tap water was varied using hydrochloric acid and sodium hydroxide to achieve pH ranges from 1 to 13 (Lentz et al., 1985). Three clays were tested using these permeants at gradients of 400–500, all without any detrimental effect. Since acids would be expected to result in permeability increases it is necessary to question whether these data are an artifact of the test conditions. Jefferis (1992) warns that tests must be carried out for sufficient time to allow for the full effects of reaction, particularly where relatively few pore volumes are permeated under high gradients. An examination of Fig. 3.23 demonstrates that, if the test is stopped before the time for full reaction, the results can be quite misleading. The investigators do note that dissolution of the octahedra is possible with strong acids and dissolution of the silica tetrahedra is possible with strong bases. Consistent with this expectation, Gipson (1985) found that permeation of bentonite–silty sand mixtures with acidic leachate high in calcium and other inorganics resulted in hydraulic conductivity increases with time. The naturally occurring clayey soils did not exhibit the same hydraulic conductivity increases.

Wu and Khera (1990) investigated the changes in the properties of a mixture consisting of a chemically treated bentonite and sand in a containment environment. They performed a series of tests to determine soil-chemical compatibility. Ballivy et al. (1992) discussed the effectiveness of injected cement grout under harsh environmental conditions. They described laboratory tests where the causes of grout degradation due to the effects of leachates have been investigated.

Another study (Peterson and Glendon, 1985) suggests that the buffering

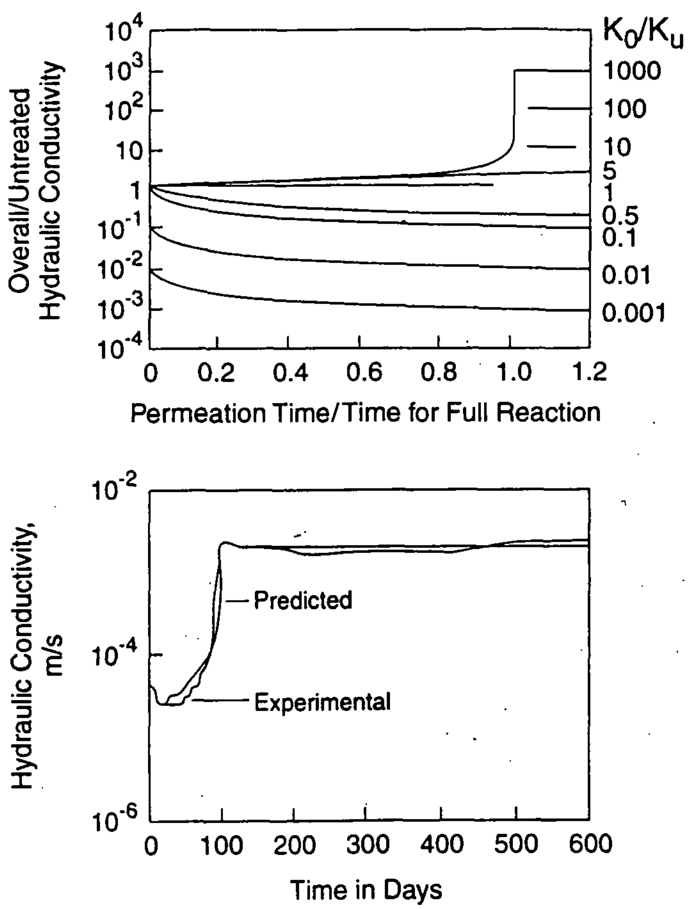

Figure 3.23 Impact of permeation time (Jefferis, 1992).

capacity of a soil may enable the permeated soil to continue to buffer acidic leachate for an extended time (in excess of 30 pore volumes). In a study testing soil-bentonite backfill, gradual and continual increases in hydraulic conductivity were observed for an acidic permeant (pH 1.0) as shown in Fig. 3.24 (Evans et al., 1985b).

While individual studies are useful in assessing the factors influencing the compatibility of clayey materials, it is also useful to consider literature reviews and syntheses of compatibility studies previously undertaken. In one such study, Mitchell and Madsen (1987) concluded the following:

1. Hydraulic conductivity changes can be understood from the perspective of the influence of chemicals on the soil fabric.
2. Chemical influences are likely to be greater with high water content ma-

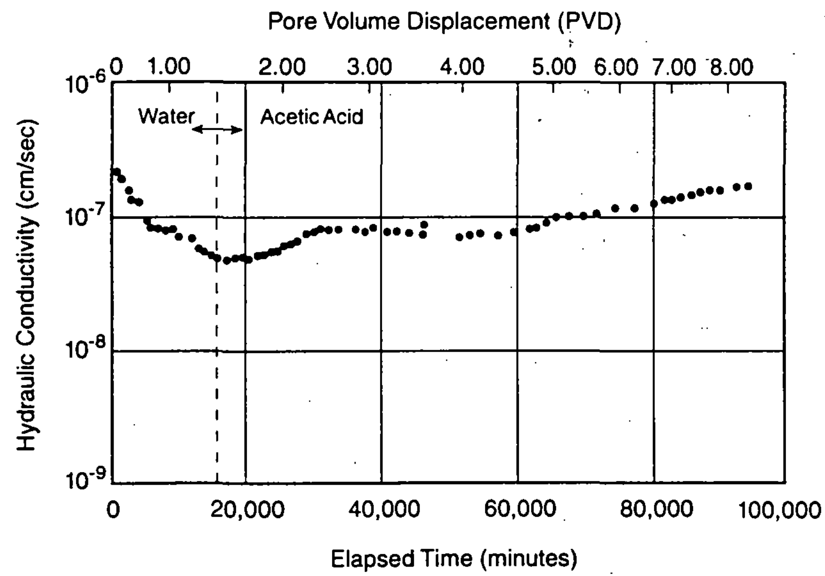

Figure 3.24 Permeation of soil-bentonite with acetic acid (Evans et al., 1985b).

terials (e.g., slurry wall backfill) than low water content materials (e.g., compacted clay).

3. Inorganic chemical effects are consistent with diffuse ion layer models.
4. Organic chemical effects are influenced by their dielectric constant, polarity, and concentration.
5. Test method selection may influence results.
6. In almost all cases, concentrated organics will cause shrinking, cracking and hydraulic conductivity increases, whereas, dilute solutions have essentially no effect.

A more recent review (Shackelford, 1994) essentially confirmed the findings presented by Mitchell and Madsen (1987). Shackelford (1993) concluded that significant increases in the hydraulic conductivity of clayey soils can result from

1. Flocculation of the clay particles due to interactions with electrolyte solutions.
2. Shrinkage of the soil in the presence of concentrated organic solvents.
3. Acid–base dissolution of the soil.

This review also revealed that there was considerable evidence supporting the use of the Gouy–Chapman theory in understanding the changes in hydraulic conductivity of clay soils. It was also noted that, for nonpolar, hydrophobic

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organic permeants, large entry pressures are required to force the permeant into the soil pores. Since field conditions do not generally result in such pressures, the interpretation of laboratory test results in the context of predicting field performance may be questionable in some cases. Nonpolar liquids tend to fill and flow only through the larger pores.

Based upon the literature described above, a number of general conclusions regarding soil–bentonite compatibility can be inferred. These conclusions are

1. Soil–bentonite backfill permeated with concentrated organic fluids is likely to exhibit increases in hydraulic conductivity as compared to permeation with water.
2. Increases in hydraulic conductivity due to permeation with concentrated organic fluids or with inorganically contaminated liquids are limited; that is, the hydraulic conductivity initially increases and then levels off to a new equilibrium value.
3. Increases in hydraulic conductivity can be limited by limiting the interactions between the soil and the contaminant. The more non-colloidal material (gravel, sand, and silt) in the soil (while still maintaining the desired hydraulic conductivity) the less will be the magnitude of hydraulic conductivity increase.
4. Permeation of soil–bentonite backfill with strong acids or bases may cause dissolutioning of the soil skeleton causing increases in hydraulic conductivity, which continue as long as the soil is exposed to fresh acids or bases.
5. Soils have a buffering capacity that may delay the increase of hydraulic conductivity when permeated by acids or bases.

Based on the available data it appears that the probability for incompatibility between the subsurface contaminants and the vertical barrier materials is low. The probability for degradation in cutoff walls increases in zones or portions of the wall where the organic constituents are in NAPL form. In this scenario, DNAPL could be found at the base of an aquifer or LNAPL could be found floating on the groundwater and, in both cases, the NAPL could be in contact with the cutoff wall. In these localized areas (where NAPL is in direct contact with the cutoff wall) some degradation is possible. An anticipated inward gradient typical of most pump-and-treat options would act to mitigate the potential for degradation under these conditions and for escape of pollutants if degradation did occur. Where the wall is not in contact with NAPL, the results of the published literature indicate compatibility problems would not be expected. If no NAPL is anticipated, the probability for incompatibility can be expected to be quite low.

Laboratory investigation programs can be designed to provide site-specific evidence of the compatibility or incompatibility between vertical barrier materials and contaminants in the vicinity of the cutoff wall. Concerns regarding incompatibility would be alleviated for any given project, should the findings from these tests be negative.

3.6 SOIL–BENTONITE BACKFILL DESIGN

In order of decreasing importance for environmental applications, the desired characteristics for a soil–bentonite backfill include:

1. Chemical compatibility
2. Low permeability
3. Low compressibility
4. Moderate strength

The most important soil parameters which affect these characteristics are grain size distribution and the water content of the backfill. Soil–bentonite backfills have been successfully created from materials varying from clean sand to highly plastic clay. However, for the containment of hazardous waste, backfill requirements are necessarily more stringent than those used for conventional barrier applications. The difference in requirements results from two fundamental differences in expected performance. First, conventional dewatering applications may not require the same degree of "perfection" as these systems are temporary. Small leaks may be inconsequential from a dewatering standpoint whereas from a hazardous waste containment standpoint they may be significant. Second, since these dewatering applications are short term, considerations of long-term changes in the hydraulic conductivity in the cutoff wall are not necessary. For hazardous waste containment applications, the long-term permeation of the cutoff wall with contaminants may alter the permeability of the material. Thus, compatibility is very important.

For waste containment applications the backfill must be designed to minimize any potential changes in hydraulic conductivity. Generally, a lower hydraulic conductivity is required for hazardous waste containment applications than for conventional dewatering applications. These goals of long-term stability coupled with low hydraulic conductivity can be best achieved by fabricating the backfill from a well-graded soil blended with soil–bentonite slurry. This well-graded soil should contain all particle sizes, including coarse, medium, and fine gravel; coarse, medium and fine sand; silt; and clay. A recommended particle size range is shown in Fig. 3.25. Since the coarser granular materials in a well-graded backfill material are in point-to-point contact, a relatively low compressibility results. Furthermore, with the well-graded nature of the material, the pore sizes (which become progressively finer) are filled

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February 1984

SLURRY TRENCH CONSTRUCTION

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continues until the bentonite is fully hydrated, which can take as long as a full week (Boyes 1975).

2.1.2.2 Dispersion

The surfaces of the clay particles in bentonite are predominately negatively charged. When two of these clay surfaces are in close proximity to one another, they repel each other due to long-range coulombic forces (Mustafa 1979). The causes of this repulsion will be discussed in Section 2.1.3.1. The effect of this repulsion is that the clay particles remain for the most part dispersed throughout the slurry. This dispersion allows the intimate mixture of bentonite and water to be maintained.

2.1.2.3 Thixotropy

When a mixture containing 5 percent by weight bentonite and 95 percent water is allowed to stand undisturbed for a few minutes, it changes from a viscous solution to a gel-like substance. When agitated or vibrated, the gel reverts to a slurry. The gel will reform each time the agitation ceases. This behavior is the result of a property called thixotropy.

Thixotropy is important in slurry trench construction because the gel structure is what keeps the particles of trench spoils in suspension in the slurry.

Thixotropy is measured by determining how strong of a gel structure is formed over a set period of time. As the strength of the gel structure increases and the speed of gel formation increases, the degree of thixotropy is said to increase. The strength of the gel structure (called the gel strength) is measured using a Fann Viscometer. Measurements are taken at 10 seconds and 10 minutes. In a high quality bentonite, the 10-minute gel strength should be only slightly higher than the 10 second gel strength (Boyes 1975).

[redacted]

Because bentonite is a natural, rather than manmade substance, its quality, and therefore its performance, is likely to vary from deposit to deposit. Several factors influence the performance of bentonites in slurry trench construction. These factors include:

- Montmorillonite content and properties
- Relative sodium and calcium concentrations

- Fineness of grinding of the raw material
- Chemical additives.

2.1.3.1 Montmorillonite Content

As mentioned previously, bentonite contains about 90 percent montmorillonite and 10 percent impurities (Boyes 1975). Montmorillonite, or smectite, is the crystalline material that gives bentonite its unique properties. To understand the behavior of this mineral, it is necessary to know its general structure and some of the interactions between montmorillonite crystals, water molecules, and cations. A description of montmorillonite structure is given below, followed by a detailed discussion of clay-water and clay-cation interactions as they affect the physical properties of montmorillonite.

a. Montmorillonite Crystal Structure

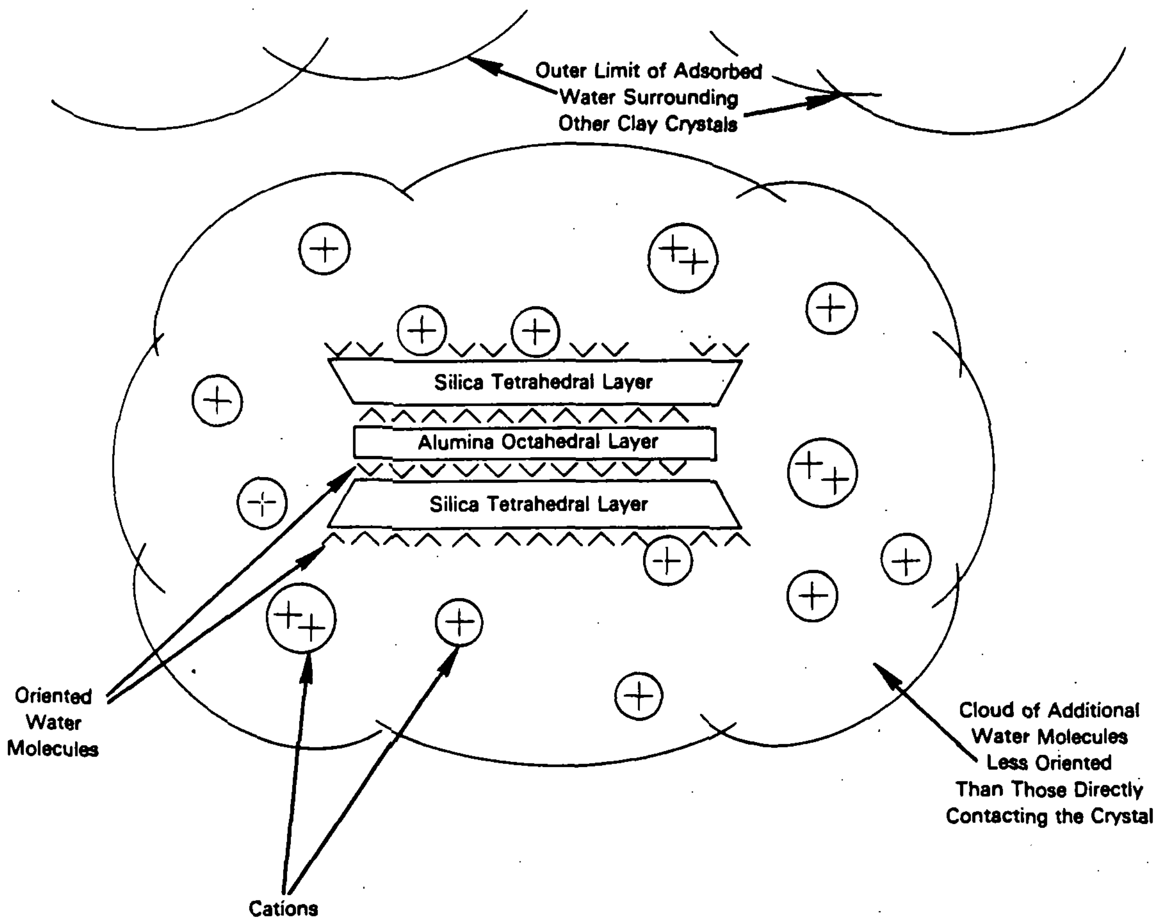
Crystals of this clay are composed of three distinct layers, as shown in Figure 2-1. The outer layers are a tetrahedral arrangement of silicon and oxygen molecules. Some of the silicon atoms in these layers have been replaced by aluminum. Sandwiched between the silica layers is a layer of aluminum atoms surrounded by six hydroxyl or oxygen atoms in an octahedral shape. Some of the aluminum atoms in this layer have been replaced by magnesium. Because of the substitutions in the three layers, unsatisfied bonds exist within the crystal, resulting in a high net negative charge. To satisfy this charge, cations and water molecules are adsorbed onto the internal and external surfaces of the clay crystals. These surfaces comprise the exchange complex of the clay. The types of cations adsorbed on the exchange complex have a great influence on the properties of the clay (Brady 1974).

The characteristics of bentonite slurries are caused to a large extent by the properties of the montmorillonite they contain. As described previously, three sets of properties are particularly relevant to slurry function. These are:

- Degree of hydration and swelling
- Flocculation and dispersion characteristics
- Gel strength and thixotropy.

The extent to which these montmorillonite properties are expressed varies considerably, depending on the types of cations adsorbed to the surface of the clay. Although numerous cations and organic molecules can be adsorbed, two cations are of primary interest in slurry trenching situations. These are sodium and calcium.

Figure 2-1.
Montmorillonite Crystal Lattice, Showing Adsorbed Cations and Oriented Water Molecules



Note: Not to Scale
Source: Based on Grim, 1968

Sodium-saturated montmorillonites behave quite differently than the calcium-saturated varieties. These differences are summarized in Table 2-1. Theories governing the reasons for these differences are described in detail below.

b. Theory of Clay Hydration and Swelling

During hydration of montmorillonite, water molecules are adsorbed to the clay crystal surface by the attraction between the hydrogen atoms on the water molecules and the hydroxyls or oxygens on the outer clay surface and in between the silicate layers. This is illustrated in Figure 2-1. The adsorbed water is held so strongly by the clay that it may be thought of as a non-liquid, or a semi-crystalline substance. Even the water molecules that do not directly contact the clay surface are influenced by the montmorillonite crystals. This is because the water molecules that are bonded to the clay surface form partially covalent bonds with a second layer of molecules. In addition, the second layer of water molecules forms partially covalent bonds with a third layer, which bonds to a fourth layer, and so on. The water in these layers surrounding the crystal surface is oriented, forming what may be thought of as a semi-rigid structure (Grim 1968).

The number of layers of water molecules and the regularity of their configuration is dependent upon the types and concentrations of cations associated with the clay. The cations tend to disrupt water adsorption, and the degree of disruption depends on the size of the hydrated cation, its valence, and its tendency to disassociate with the clay surface during hydration (Grim 1968).

Sodium ions disrupt hydration much less than calcium ions. For example, sodium-saturated montmorillonites have been found to influence the orientation of water molecules more than 100 Angstroms from their crystal faces. This corresponds to about 40 molecular layers of water. In contrast, calcium-saturated montmorillonites have much smaller spheres of influence, on the order of 15 Angstroms, or about 6 molecular layers of water (Grim 1968).

The observable effects of these sub-microscopic interactions are that sodium montmorillonites adsorb much more water and swell far more than do calcium montmorillonites. As a result, as the amount of sodium on the exchange complex of montmorillonite increases, the amount of swelling increases (Rowell, Payne and Ahmad 1969). In addition, a 5 percent solution of highly hydrated sodium montmorillonite has a much higher viscosity than a 5 percent calcium montmorillonite solution. In fact, a 5 percent solution of sodium bentonite in water can exhibit a viscosity of 15 centipoise, but it takes 12 percent calcium montmorillonite in a solution to obtain the same viscosity (Grim and Guven 1978). This is illustrated in Figure 2-2.

TABLE 2-1

COMPARISON OF SODIUM AND CALCIUM-SATURATED MONTMORILLONITES

Parameter	Sodium-Saturated Montmorillonite	Calcium-Saturated Montmorillonite
Swelling upon hydration, cm^3/g of clay	11 (1)	2.5 (1)
	(Wyoming sodium bentonite)	(4 base-exchanged bentonites tested)*
Hydration rate, 5% solution (2)	Hydrated to ~9cP in 10 min., stabilized at 9.2cP by 20 min. 3% solution of polymer treated sodium bentonite hydrated to 17.2cP in 10 min., then stabilized.	hydrated to ~13cP in 10 min., stabilized at ~14 to 18 cP in 4 hours.*
Cation exchange Capacity, meq/100g.	80-150 (3)	60-100 (2)
Degree of thixotropy	high (2)	low (6)
Liquid limit	300-700 (4, 5)	155-177 (4)
Plastic Limit (4)	75-97	65-90
Yield in barrels of 15cP drilling mud per ton of clay (4)	125	18-71
Percentage of clay by weight in water to produce a 15cP colloidal suspension (4)	~5	~12

TABLE 2-1

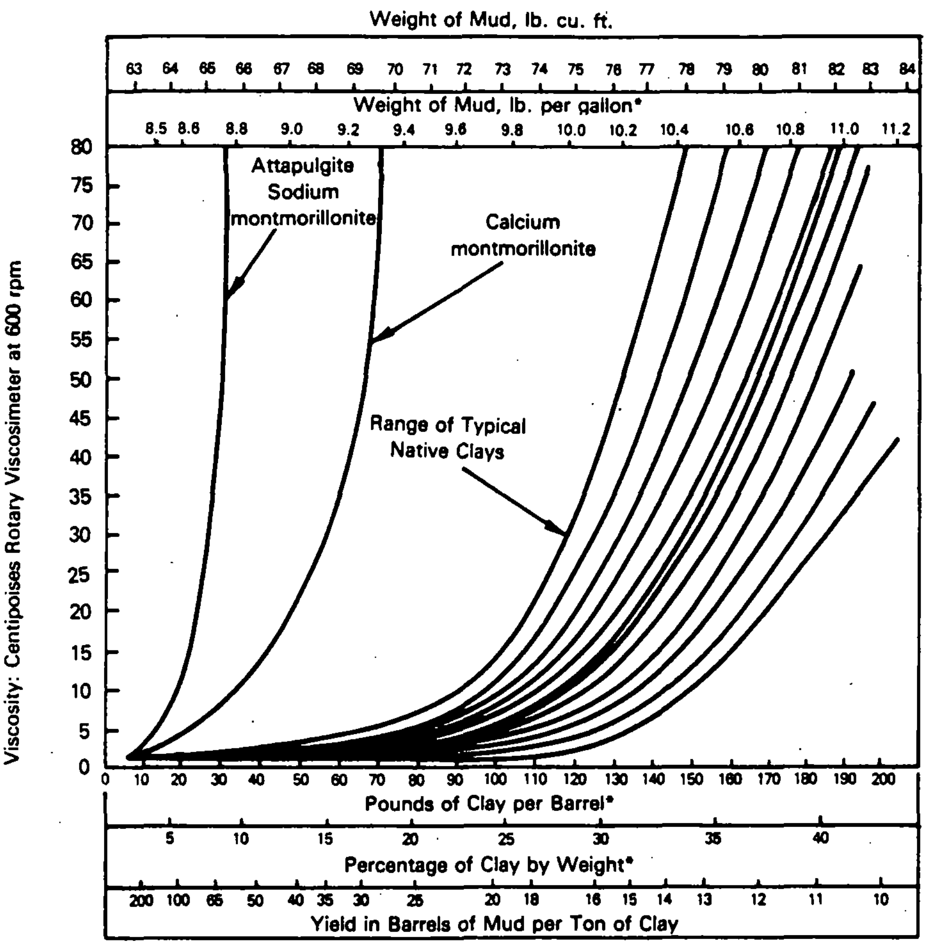
COMPARISON OF SODIUM AND CALCIUM-SATURATED MONTMORILLONITES

Parameter	Sodium-Saturated Montmorillonite	Calcium-Saturated Montmorillonite
Permeability of a 9:3 quartz to clay mixture (cm/sec) (4)	2.76×10^{-9}	7.2×10^{-7}
Permeability of a 7:3 quartz to clay mixture (cm/sec) (4)	5.0×10^{-10}	3.5×10^{-8}

*Base-exchanged bentonites are calcium bentonites that have been treated with sodium compounds to increase their adsorbed sodium content. They are commonly used in European slurry trenching construction (Boyes 1975).

References: (1) Baver, Gardner and Gardner 1972, (2) Boyes 1975, (3) Grim 1968 (4) Grim and Guven 1978, (5) Xanthakos 1979, (6) Case 1982.

Figure 2-2.
Viscosity and Weight of Mud in Relation to Percentage of Bentonites and Native Clays in Fresh Water



*Clay Specific Gravity Assumed to be 2.50.

c. Theory of Flocculation and Dispersion

Adsorbed cations also influence the flocculation and dispersion of colloidal clay suspensions. This is relevant to slurry wall construction in that ions in the groundwater and calcium ions in cement strongly affect slurry properties.

Montmorillonite crystals that are saturated with calcium ions have smaller spheres of influence than sodium-saturated types. This is thought to occur because the larger divalent calcium ions are held more strongly to the clay, thus the effective net negative charge on each clay particle is lowered, and the size of the diffuse double layer surrounding each clay particle is reduced. The diffuse double layer is a swarm of cations and water molecules near the surface of the clay particle, surrounded by a layer of anions that are attracted to the cations. The concentration of the cations decreases as one moves away from the clay surface. The diffuse double layer acts as a buffer between clay particles (Baver, Gardner and Gardner 1972). As shown in Figure 2-3, clay faces exhibit a net negative charge, while the edges have a positive charge. This results in a repulsion between the crystal faces but an attraction between edges and faces.

When sodium is the dominant cation on the clay surface, the diffuse double layer is extensive and the colloids are well dispersed throughout the water. Very little face-to-face contact occurs. When calcium is present in sufficient quantities, the double layer is constricted and the water molecule orientation is severely reduced. Thus, the repulsion between clay crystals is reduced, face-to-face contact can occur, and the particles can form "packets," or "flocs." (See Figure 2-3.) The formation of flocs is called flocculation, and this process reduces the amount of swelling that occurs and the viscosity of the solution (Baver, Gardner and Gardner 1972; Boyes 1975).

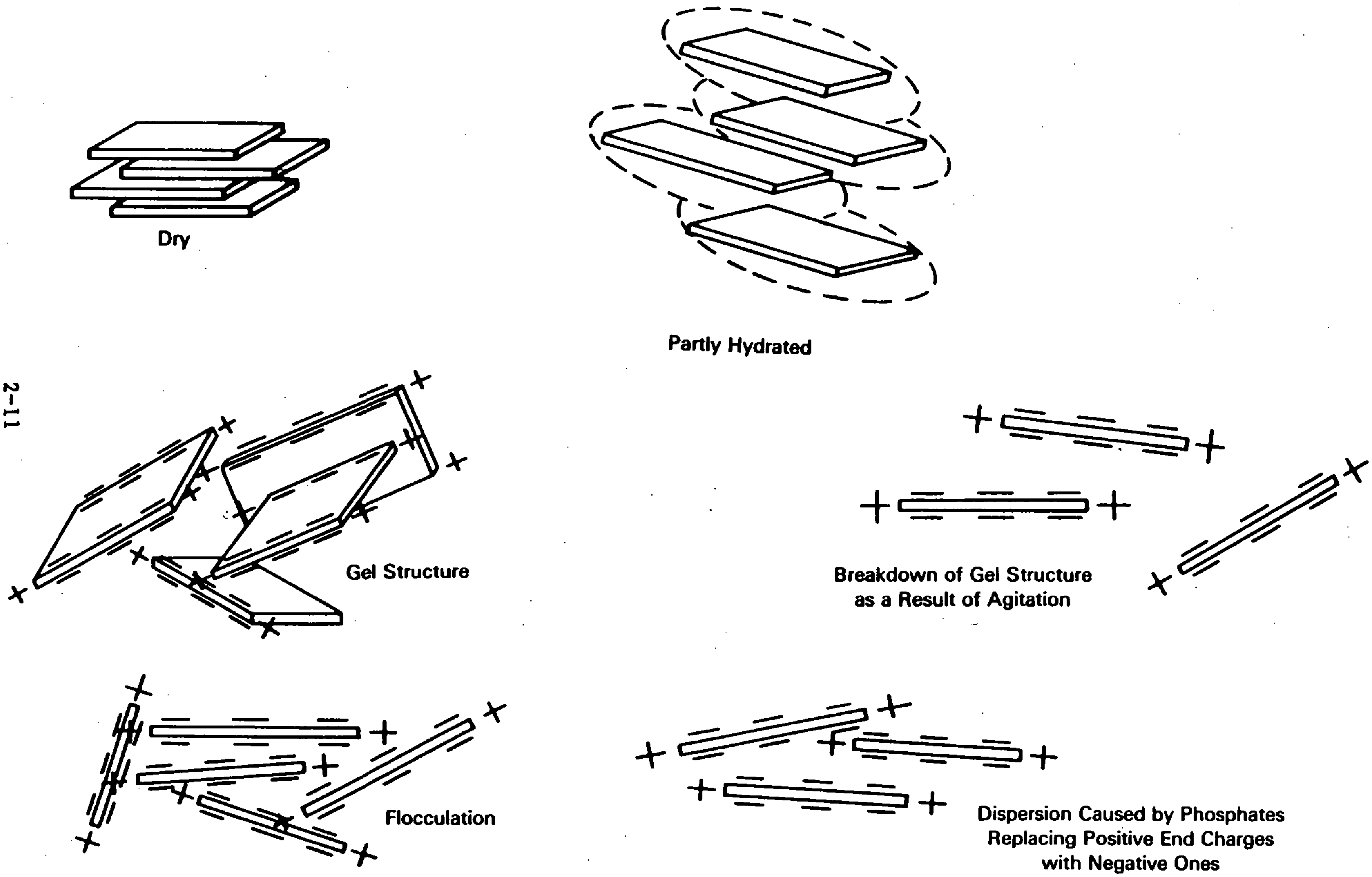
One of the observable effects of flocculation on slurries and slurry walls is a substantial increase in permeability. When the zone surrounding each particle is constricted, the amount of swelling is reduced and voids are created. Through these voids, solution movement can and does occur. As shown in Table 2-1, the permeability of a mixture of 7 parts quartz sand and 3 parts sodium montmorillonite was measured at 5.0×10^{-10} cm/sec, while the same mixture using calcium montmorillonite had a permeability two orders of magnitude higher, or 3.5×10^{-8} cm/sec. In slurries, when flocculation occurs, the flocs can become large enough to begin settling out of the suspension (Boyes 1975). This can reduce trench stability and interfere with filter cake formation, as discussed earlier in this section.

d. Theory of Gelation and Thixotropy

One of the most interesting and useful properties of montmorillonite suspensions is thixotropy. This property is the ability of the colloidal suspension to thicken, or gel upon standing, become less viscous when agitated, yet re-gel when agitation ceases. It is caused by the formation of

Figure 2-3.

Bentonite Particles During Hydration, Gelation, Flocculation, and Dispersion



Source: Boyes, London 1975

a "house of cards" structure between positively charged clay particle edges and negatively charged clay faces, as illustrated in Figure 2-3 (Xanthakos 1979). In practice, the gelation of the bentonite slurry provides support for small particles of soil to remain in suspension rather than to sink to the trench bottom (Boyes 1975).

The amount of thixotropy is determined by measuring the gel strength of the slurry. The gel strength is "the stress required to break up the gel structure formed by thixotropic buildup under static conditions" (Boyes 1975). It is measured using a Fann viscometer, as described in Section 4. The difference between the gel strength 10 seconds after agitation and the gel strength after standing for 10 minutes is a measure of the slurry's thixotropy (Xanthakos 1979).

Measurements of 10 minute gel strengths of bentonite slurries can range from about 5 to 20 lb/ft^2 and average 10 to 15 lb/ft^2 (Xanthakos 1979).

The bentonites used during slurry trench construction behave essentially like the sodium saturated montmorillonites described above. The properties of hydration, flocculation, dispersion and gel strength that are exhibited by the slurries are a result of the interactions of montmorillonite crystals, water molecules, and cations. The ability of a bentonite slurry to perform its functions during slurry trench construction is dependent on these interactions.

2.1.3.2 Relative Sodium and Calcium Concentrations

Natural sodium bentonite from Wyoming is commonly used in many of the slurry trenching operations in the United States. These bentonites do not contain pure sodium montmorillonite. One bentonite was reported to contain 60 percent sodium on its exchange complex, with the remaining sites being held by calcium and magnesium. However, the average distribution of cations on Wyoming bentonite is somewhat different. Most of the Wyoming bentonite currently being sold contains an average of 38 to 50 percent sodium, 15 to 35 percent calcium and 10 to 30 percent magnesium (Alther 1983).

High sodium bentonites should be more effective than the low sodium grades in many situations. At sites where a high concentration of calcium salts occurs in the soil or groundwater, or where cement bentonite slurries will be used, higher sodium bentonites are particularly recommended, for the reasons described below. The detrimental influence of the calcium from the cement or the groundwater on the sodium bentonite can be substantial. This is due to the strong attraction between calcium ions and montmorillonite crystals. Because this attraction is so strong, calcium ions can easily displace sodium ions on the clay. The ease of replacement of sodium by calcium increases as the concentration of calcium in the solution and on the clay surface increases. After about 30 percent of the exchange sites on the clay surface become occupied by calcium, the bentonite acts more like calcium montmorillonite than the sodium variety (Grim 1968).

Because there are limited quantities of natural sodium bentonites, some areas are forced to use specially treated calcium bentonites instead. This occurs most frequently in Europe. These calcium bentonites are exposed to sodium-containing materials such as sodium hydroxide to force some of the calcium ions off of the exchange complex of the montmorillonite and then replace them with sodium ions (Grim 1968). Sodium carbonate, which is less expensive and more effective than sodium hydroxide, is also used on some bentonites (Alther 1983). As long as there is less than 30 percent calcium and at least 50 percent sodium on the exchange complex of the montmorillonite, the material will act essentially like a sodium montmorillonite (Grim 1968; Shainberg and Caiserman 1971).

2.1.3.3 Bentonite Particle Size

This purely physical parameter can influence the performance of the bentonite in a number of ways. Finely ground bentonite has a larger surface area per unit weight than coarser bentonite because as particle size decreases, surface area per unit weight increases. The increased surface area of the finer particles allows the bentonite to hydrate more readily and form a gel structure more quickly than coarser particles of the same bentonite. Thus the average particle size of the bentonite can affect its performance in the slurry. Typically, the types of bentonite that are recommended for slurry trenching have been pulverized to yield particles small enough so that 80 percent will pass through a number 200 mesh sieve (Federal Bentonite 1981).

2.2 Bentonite Slurries

The Wyoming bentonites most commonly used in slurries are mixed at a rate of from 4 to 7 percent bentonite in 93 to 96 percent water (Boyes 1975). This muddy mixture stabilizes the sidewalks of the open trench during excavation. The properties of a well-functioning slurry and the factors that affect bentonite slurry quality are discussed below.

2.2.1 Bentonite Slurry Properties

To maintain trench stability while exhibiting suitable flow characteristics, the slurry must have the proper viscosity, gel strength and density. It must form a thin, tough, low-permeability filter cake rapidly and repeatedly. The bentonite slurry supplied to the trench may meet or exceed the quality standards stated in the specifications, however, slurry properties are altered during trench excavation and slurry quality may either improve or degrade during use. Table 2-2 presents data on fresh and in-trench slurries. As shown in this table, the density, viscosity, gel strength, and solids content of the slurry generally increases during excavation, while the overall water content decreases, due to the increased solids content. Brief

2.2.2.4 Chemical and Physical Additives

Numerous chemical and physical additives have been used in slurries to improve their viscosity, gel strength, density, or fluid loss rate (Xanthakos 1979). Some of those additives are listed in Table 2-3. It is recommended that the use of any slurry additives be allowed only with the approval of the engineer. Some slurry trench excavation specifications forbid the use of chemically treated bentonites (U.S. Army Corps of Engineers 1975). One problem with the use of chemically treated bentonites is the possibility of enhanced interaction with pollutants. Conversely, certain chemical treatments may render the bentonite less susceptible to chemical attack. Slurry/waste interactions are discussed in Section 4.

2.3 Soil-Bentonite Walls

SB walls are excavated under a bentonite slurry in a continuous trench. As excavated materials are removed from the trench, they are mixed with slurry and replaced in the trench a short distance from the active excavation area. Techniques used during slurry trench construction are described in detail in Section 5.

2.3.1 SB Wall Properties

A properly designed and constructed SB wall exhibits the following properties:

- Low Permeability
- Resistance to hydraulic pressure and chemical attack
- Low bearing strength and moderate to high plasticity.

2.3.1.1 Low Permeability

Permeabilities of completed soil-bentonite cut-offs have been as low as 5.0×10^{-9} cm/sec, although higher permeabilities are more common (Xanthakos 1979). Typical permeabilities of SB walls range from over 10^{-5} cm/sec in walls composed primarily of coarse, rather than fine materials, to less than 10^{-8} cm/sec in walls containing over 60 percent clay (D'Appolonia 1980b).

TABLE 2-3

COMMON SLURRY MATERIALS AND ADDITIVES

Weight materials	Barite (barium sulfate) or soil (sand)
Colloid materials	Bentonite (Wyoming, Fulbent, Aquagel, Algerian, Japanese, etc.), basic fresh water slurry constituent Attapulgite, for saltwater slurries Organic polymers and pretreated brands
Thinners and dispersing agents	Quebrancho, organic dispersant mixture (tannin) Lignite, mineral lignin Sodium tetraphosphate Sodium humate (sodium humic acid) Ferrochrome lignosulfonate (FCL) Nitrophemin acid chloride Calcium lignosulfonate Reacted caustic, tannin (dry) Reacted caustic, lignite (dry) Sodium acid pyrophosphate Sodium hexametaphosphate
Intermediate-sized particles	Clay, silt, and sand
Flocculants and polyelectrolytes	Sodium carboxymethyl cellulose (CMC) Salts Starches Potassium aluminate Aluminum chloride Calcium
Fluid-loss-control agents	CMC or other flocculants Pregelatinized starch Sand in small proportions
Lost-circulation materials	Graded fibrous or flake materials; shredded cellophane flakes, shredded tree bark, plant fibers, glass, rayon, graded mica, ground walnut shells, rubber trees, perlite, time-setting cement, and many others.

Reference: Xanthakos 1979. Copyright 1979 by McGraw-Hill Books. Used with Permission.

2.3.1.2 Resistance to Hydraulic Pressure and Contaminants

An SB wall that exhibits an extremely low permeability is not effective in the long run if it cannot withstand the hydraulic gradients induced by its presence or if it disintegrates upon contact with contaminants at the site.

Because of its low permeability, the wall can be used to severely restrict downgradient water movement. This causes the water level on the upgradient side of the wall to rise significantly as compared to the downgradient side. This difference in water levels is termed the hydraulic gradient. A high hydraulic gradient across the wall is likely to develop unless groundwater rerouting is accomplished through the use of upgradient extraction wells, subsurface drains or interceptor trenches (see Section 3). Despite the use of these ancillary measures, the wall should be designed to withstand significant hydraulic gradients. The incorporation of a high concentration of clayey materials into the backfill improves the wall's long-term resistance to hydraulic gradients up to 200 (D'Appolonia 1980b). Wall design is discussed in Section 5.

The wall's resistance to degradation by chemical contaminants is also a primary measure of long term performance. Prior to SB wall construction, extensive testing of the effects of the site's leachate on proposed backfill mixtures should be conducted. In general, clayey backfill mixtures withstand permeation with contaminants more effectively than those that contain less clay (D'Appolonia 1980b).

2.3.1.3 Strength and Plasticity

The strength of SB cut-off walls is not usually of primary concern when designing pollution migration cut-offs. These walls are usually designed to be comparable in strength to the surrounding ground (Jefferis 1981b). If stronger walls are required, coarser material may be added to the backfill, although this practice results in an increase in wall permeability (Millet and Perez 1981). In any case, the strength of a soil-bentonite wall is not usually relevant in hazardous waste applications, except where traffic must pass over the wall. Design of traffic caps is discussed in Section 5.

The response of the SB wall to lateral earth pressures and earth movements is an important factor in the design of pollution migration cut-offs. If the wall is too brittle, shifts in nearby strata caused by overloading the surface by stockpiles or heavy machinery can result in cracking and subsequent leakage of the wall. Fortunately, completed SB cut-off walls behave plastically when stressed. That is, they undergo plastic deformation rather than crack (Guertin and McTigue 1982b). In contrast, CB walls have higher strength than SB walls and can be brittle and thus more easily cracked (Millet and Perez 1981).

2.3.2 Factors Affecting SB Wall Performance

There are numerous factors that can affect the performance of SB Walls. These can be divided into four general groups which are:

- Design criteria
- Backfill composition and characteristics
- Backfill placement methods
- Post-construction conditions at the site.

2.3.2.1 Design Criteria

The design criteria that affect SB wall performance include wall width, wall depth, selection of appropriate aquiclude, wall configuration, and use of ancillary measures. These criteria are discussed in Section 5. The factors relating to backfill preparation and post-construction conditions are described below.

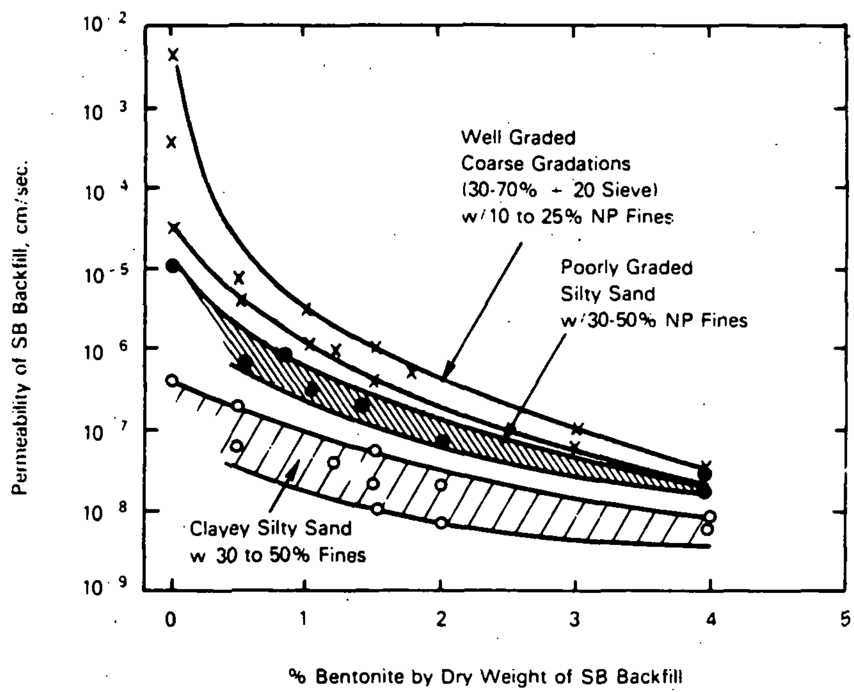
2.3.2.2 Backfill Composition and Characteristics

To produce a low-permeability, durable cut-off wall, the backfill must contain a high concentration of plastic fines (clays), a minimal amount large-diameter particles, and a suitable concentration of bentonite and water. Contaminants in the soil or water can also affect the wall's performance.

a. Native Clay and Bentonite Content

A primary requirement for backfill material is that it contain a suitable particle size distribution. For low permeability, this means the backfill must have from 20 to 40 percent fine particles, preferably plastic fines. Fine particles (less than 0.074 mm in diameter or passing a number 200 sieve) exert a significant influence on backfill permeability, as shown in Figure 2-9. At a given bentonite concentration, the backfill permeability will be lower when the backfill material contains a higher proportion of fines. Conversely, increasing the bentonite content of the backfills tested significantly reduced the wall permeability. The bentonite content of the mixed backfill should not fall below 1 percent (D'Appolonia 1980b). Where the strength of the cut-off wall is of primary concern, a higher concentration of coarse and medium sized particles are required. In any case, material over 6 inches in diameter are not considered desirable for use in backfills (Federal Bentonite 1981).

Figure 2-9.
Relationship Between Permeability and Quantity of Bentonite Added to SB Backfill



Source: D'Appolonia, 1980

D'Appolonia (1980b) found that plastic fines reduce permeability more effectively than nonplastic fines. This is most likely due to the fact that plastic fines are composed of smaller particles than nonplastic fines. The effect of plastic fines on backfill permeability is shown in Figure 2-10.

Fine particles, particularly clays, contribute to low permeability by assisting in bridging the pores between larger particles and by contributing to the swelling, viscosity, gelation, and cation exchange capacity of the backfill (D'Appolonia 1980b, Boyes 1975). Although these properties find their maximum expression in montmorillonite, other clays exhibit these characteristics to a lesser degree (Grim 1968). Thus the clay content of the backfill has a pronounced effect on SB wall permeability.

b. Water Content

The water content of the backfill can also influence the SB wall performance. The amount of water in the backfill should be carefully controlled because the hydraulic conductivity of sodium montmorillonite has been reported to increase dramatically as the water content increases (Low 1976). There is an effective limit on reducing the water content of the backfill, however, because the backfill must slump sufficiently to allow proper placement. The water content of backfills at ideal slumps is from 25 to 35 percent (D'Appolonia 1980a). Even so, the excess water in the backfill has been found to result in increased permeability (Jefferis 1981b).

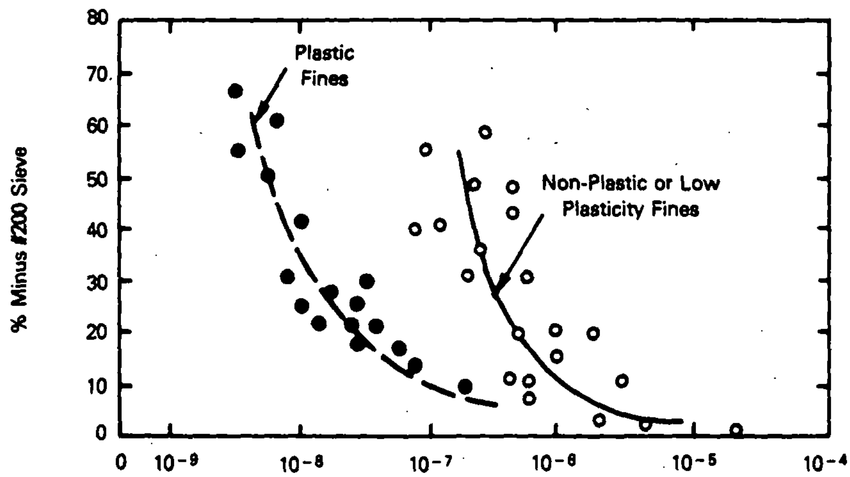
If the moisture content of the soil material excavated from the trench is over 25 percent initially, the addition of bentonite slurry during backfill mixing results in a very wet backfill that exhibits high permeability. To remedy this situation, D'Appolonia (1980a) suggests spreading the soil material in a thin lift over the backfill mixing area, then broadcasting dry bentonite over the lift at the desired rate. The soil material is then mixed with the dry bentonite prior to the addition of the slurry. This reduces the water content of the backfill while simultaneously increasing the bentonite content.

[redacted]

The construction of a low-permeability SB cut off walls requires the use of soils in the backfill that are free of deleterious materials. To be free of deleterious materials, the proposed soil source must not contain significant amounts of soil organic matter, including plant and animal debris, high calcium materials, including gypsum, chalk and caliche, or high concentrations of soluble salts, including sodium chloride, sodium sulfates or anhydrite.

In addition to the items listed above, other subsurface materials may be detrimental to backfill quality. For example, at some sites where pollution migration cut-offs have been constructed, the soil excavated was contaminated with pollutants. These pollutants may or may not significantly interfere with

Figure 2-10.
Effect of Plastic and Non-plastic Fines Content on Soil-Bentonite Backfill Permeability



Source: D'Appolonia, 1980

cut-off wall performance. D'Appolonia (1980a) suggested preparing a test mixture to determine compatibility. He further suggested using the contaminated soil if equal in quality to uncontaminated soil, even though the material may decrease the slurry and backfill performance initially. This is because early exposure of the bentonite to the contaminants reduces the permeability changes that occur during subsequent exposure to the contaminants. This approach must be balanced against the fact that contaminant breakthrough may occur earlier.

2.3.2.3 Backfill Placement Methods

The mixing and placement of the carefully selected backfill material is of critical importance in the overall performance of the completed wall. The bentonite slurry and soil material must be combined to form a relatively homogenous paste with a consistency similar to that of mortar or concrete. It must flow easily yet stand on a slope of about 10:1, and must be at least 15 pcf (240 kg/m^3) denser than the slurry in the trench (D'Appolonia 1980b). The methods used to mix the backfill and the tests used to measure its shear strength, flow characteristics and density are described in Section 5.

2.3.2.4 Post-construction Conditions

Once the backfill has been mixed and placed, the performance of the wall is dependent on the subsurface conditions surrounding the wall. In particular, the hydraulic gradient and the presence of contaminants can influence the wall's ability to function properly.

a. Hydraulic Gradient

The difference in hydraulic pressure between the upgradient and downgradient sides of the trench strongly influences the trench's durability as well as its initial permeability. Little data are available on this factor; however, it has been shown that high hydraulic pressures within the trench during filter cake formation result in a lower permeability filter cake. The long-term effect of high hydraulic pressure differentials across the trench on wall permeability is, however, likely to be different (D'Appolonia 1980b). A large difference in hydraulic pressure from one side of the trench to the other is expected to severely tax the integrity of the wall. Methods used to combat high hydraulic gradients include increasing wall thickness and/or using extraction wells or subsurface drains upgradient to assist in equalizing hydraulic pressures near the wall. These are discussed in Section 5.

b. Presence of Contaminants

The resistance of soil-bentonite cut-off walls to permeation and destruction by various pollutants is the subject of much current research. Bentonite is extremely resistant to degradation from some substances, but others cause rapid dehydration and shrinkage of the montmorillonite particles. SB wall performance can be severely inhibited by contact with incompatible chemical compounds in leachates or wastes.

The wall can be protected from degradation due to chemical incompatability in several ways. First, waste/wall contact can be minimized by using extraction wells or subsurface drains. Second, contaminated soil can be used in the backfill, as described earlier. Third, the concentration of non-montmorillonite clay in the backfill can be maximized.

Non-montmorillonitic native clays are not likely to be as severely affected by chemical contaminants as are bentonites or native montmorillonitic clays. This is because the non-montmorillonitic native clays do not swell as extensively as montmorillonite when they are hydrated. Consequently, if they become dehydrated during chemical interactions, they do not shrink as extensively as montmorillonite does when it becomes dehydrated. When shrinkage is minimized, the associated permeability increase is also minimized. Thus the adverse effects of the chemical interaction can be decreased.

Different types of wastes affect the clay in the backfill in different ways. In addition to dehydration and shrinkage, the clay may be dissolved or its properties can be drastically altered. Data on chemical compatabilities of wastes and SB walls are summarized in Section 4.

The proper design and construction of an SB wall can result in a durable, low permeability cut-off that withstands high hydraulic gradients and permeation with various contaminants. At some sites, the use of SB walls is not appropriate (see Section 5). When SB walls cannot be used CB walls can be installed. These walls are similar to SB walls in that they contain bentonite and form a relatively low permeability cut-off, but they differ in several important ways, as described below.

2.4 Cement Bentonite Slurries

When CB walls are being constructed CB slurries are prepared. Techniques used to construct CB walls are described in Section 5.

2.4.1 CB Slurry Properties

Cement-bentonite slurries normally contain about 6 percent by weight bentonite, 18 percent ordinary Portland cement (o.p.c.) and 76 percent water

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Environmental Protection
Agency

Hazardous Waste Engineering
Research Laboratory
Washington DC 20460

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Research and Development

Construction Quality Control and Post-Construction Performance Verification for the Gilson Road Hazardous Waste Site Cutoff Wall

completion of the cutoff wall, the levels of contaminants were highest within the bedrock.

With the completion of the wall, contaminant levels in the area of the stream again decreased. In addition, concentrations inside the cutoff wall remained higher than levels outside the cutoff wall. Contaminant levels in the vicinity of the cutoff wall were highest in the bedrock. These levels indicate flow in the vicinity of the cutoff wall was occurring within the fractured bedrock and not through the wall itself.

D. Contaminant Distribution in Response to the Permanent Recirculation System

Currently, contaminant levels within the cutoff wall have decreased as compared to levels measured subsequent to cutoff wall completion. This decrease is primarily due to the functioning of the permanent recirculation system. Pumping of groundwater within the cutoff wall has had the effect of mixing or homogenizing the groundwater. As a result, levels within the cutoff wall have dropped as highly contaminated zones were mixed with zones exhibiting lower concentrations. However, even with the homogenizing effect, levels within the cutoff wall are higher than those outside of the wall.

[redacted]

The cutoff wall installed at Gilson Road has performed as a multi-functional containment structure. Initially, the wall served as a temporary measure to impede the off-site migration of contaminants. With the start up of the permanent hydrodynamic isolation system, the cutoff wall now functions as a clean water exclusion barrier to inhibit the flux of clean water back onto the site in response to groundwater pumping. While performing the initial containment function, the wall was exposed to contaminants. As a result, the potential for chemical degradation of the wall was investigated.

Two major factors control the time required for cutoff wall degradation:

- The number of pore volume displacements required to affect a chemically mediated change in the wall hydraulic conductivity;

- The rate at which leachate flows through the wall.

Long-term hydraulic conductivity testing of the Gilson Road backfill with worst case leachate from the site indicated that displacement of two to three pore volumes are required to effect changes in the cutoff wall hydraulic conductivity (Schulze, 1984). The testing, which simulated in-situ conditions, indicated a maximum increase to twice the initial hydraulic conductivity (1.2×10^{-8} to 2.5×10^{-8} cm/sec). These results agree with literature documentation on chemical degradation of clay based barriers.

The flow of contaminated groundwater through the cutoff wall intact backfill is dependent upon the hydraulic gradient (i) across the wall and the intact hydraulic conductivity of the wall (k). The hydraulic gradient is determined by the ratio of the total head difference across the wall as compared to the width of the wall (L). The intact hydraulic conductivity of the wall was determined by hydraulic conductivity testing of cutoff wall samples (Section 2).

Flow through a porous medium is governed by Darcy's Law ($Q = kiA$) where Q is the volume of water flowing through a cross-sectional area of the medium (A). The average velocity of the flow through the medium is described by the equation $V_s = ki/n$, where n is the porosity of the material. The time required to displace one pore volume can be determined by the equation $t = W/Vs$, where W is the width of the medium through which flow is occurring.

In order to determine the time to displace one pore volume of water through the wall, certain conservative, or worst-case, assumptions were made in the selection of parameter values.

The greater the hydraulic gradient (i) across the wall, the faster the wall will degrade. The maximum head difference observed on the site was three feet. This condition only occurred during one period of particularly high precipitation without hydrodynamic controls. A minimum width of the wall would be equal to the width of the bucket used to excavate the cutoff wall trench. A 3 foot bucket was used. This results in a maximum hydraulic gradient of $i = 1$.

The greater the hydraulic conductivity of the wall, the faster degradation should occur. The hydraulic conductivity of the wall is inversely proportional to the level of stress on the wall. The stress on the wall increases with depth to a maximum of 50 pounds per square inch (psi) at the bottom of the wall (105 feet). In order to simulate worst case conditions, hydraulic conductivity testing of the cutoff wall was conducted using stress conditions corresponding to the top of the cutoff wall, or

3 to 5 psi. Although design phase laboratory testing and construction QC testing in the field yielded an average cutoff wall hydraulic conductivity of 5×10^{-8} cm/s, a more conservative value of 1×10^{-7} cm/s was used for determining pore volume displacement rates as based on work completed under phase one (Section 2) of this contract.

The porosity (n) of a material is that portion of the material not occupied by solid matter relative to its total volume. If all other factors remain constant, a lower porosity will result in a higher flow velocity, which will cause a faster degradation of the cutoff wall. Based on QC data, a conservative value of $n = 0.4$ was chosen.

A computation of the time to displace a single pore volume is determined using the equations stated previously:

$$t = \frac{W}{Vs}; \text{ where}$$

$$Vs = \frac{ki}{n}$$

Using the conservative, or worst case, values stated above, the form of the equations becomes:

$$Vs = \frac{1 \times 10^{-7} \text{ cm/s } (1)}{0.4} = 2.5 \times 10^{-7} \text{ cm/s.}$$

$$t = \frac{91.44 \text{ cm}}{2.5 \times 10^{-7} \text{ cm/s}} = 3.7 \times 10^{8} \text{ seconds;}$$

$$t = 11.6 \text{ years}$$

The time to displace one volume of pore water using worst case parameters is calculated to be approximately eleven years. However, under the average conditions which have actually occurred on the site, the time required to displace a single pore volume is approximately fifty years. As the testing of the degradation of the cutoff wall material has shown, chemical degradation of the material was complete only after two pore volume displacements, thereby doubling the likely time estimates quoted above.

It is realized that chemical degradation of the wall would begin immediately at the surface in contact with the leachate. As a result, the hydraulic conductivity of the backfill at the surface, over a thickness dx, would increase. Assuming as a worst case that the increase was infinite, then the gradient across the wall would increase commensurate with the decrease in effective wall width. Integration of the appropriate equations demonstrates that the rate of chemical degradation would increase by a factor of two as compared to that computed assuming constant gradient. The assumption of infinite increase in hydraulic conductivity due to chemical degradation, however, is conservative in that actual testing indicates only a two fold increase. As such, the increase in rate of degradation due to a changing gradient would be less than two.

Given the data and computations performed above, it is unlikely that the Gilson Road cutoff wall has undergone significant chemical degradation in the 2.5 years since its construction began. The calculation of potential cutoff wall degradation assumed a hydraulic gradient of 1. However, the operation of the hydrodynamic isolation system, in place since April 1985, has acted to balance the hydraulic heads inside and outside the wall, thereby reducing the hydraulic gradient to essentially zero. In addition, the operation of the groundwater treatment plant under construction at the site includes a treatment purge stream located outside the cutoff wall. As a result, pumping of groundwater within the wall will cause a water deficit within the cutoff wall and result in flux into the site. This flux into the site will cause relatively clean groundwater to flow through the wall, thereby reversing the degradation caused by the flow of leachate through the wall if any has actually occurred.

4.4 Conclusions

The hydraulic stress testing and contaminant migration analyses both indicate that the cutoff wall appears to be functioning as an essentially intact barrier. However, it is apparent that the fractured bedrock which forms the bottom of the containment is highly pervious and would result in a major leakage path without the hydrodynamic isolation systems incorporated in the overall containment design. The following more specific conclusions can be drawn from the Phase Three work as summarized herein.

- The three-dimensional numerical modeling of the site predicted that the bedrock located at the downgradient portion of the containment was more pervious than initially indicated via packer testing data. The bedrock pumping test verified this prediction, yielding bedrock hydraulic conductivities in the range of 10^{-1} cm/sec. This value is large as compared to that specified for the cutoff wall (1×10^{-7} cm/sec). The pumping test also demonstrated that the glacial till existing just above the bedrock was also quite pervious and probably discontinuous. This data supports conclusions reached during the RI/FS.

- The cutoff wall and bottom aquitard/aquifer units form a hydraulically coupled system. Analysis of cutoff wall bulk hydraulic conductivity therefore must rely on numerical modeling to correlate the stress test data and separate the behavior of the wall from that of the bedrock. The accuracy of the bulk hydraulic conductivity computed via the sensitivity analysis is therefore inherently limited to the calibration accuracy of the numerical model. As such, post-construction verification efforts are based on somewhat circumstantial data in the form of piezometric head distributions and thus could be opened to varying interpretations.

- Analyses of cutoff wall bulk hydraulic conductivity must be based on hydraulic stress testing of the containment. The value obtained will therefore inevitably be in the form of an upper bound solution. The proximity of the upper bound value obtained via analysis to that specified for the cutoff wall will be limited by not only the success of the cutoff wall construction effort, but also by the hydraulic conductivity of the containment bottom. The very cases which are likely to require post-construction verification studies (containment leakage) are therefore those for which hydraulic stress analysis may be the least conclusive. In these instances, specifications based solely on performance criteria may prove difficult to enforce.

- The bulk hydraulic conductivity of the Gilson Road cutoff wall was found to be less than 10^{-5} cm/sec. The degree to which the actual value falls below this upper bound cannot be determined due to the high hydraulic conductivity of the containment bottom. However, a worst case value of 10^{-5} cm/sec yields a cutoff wall efficiency of greater than 93%. The actual value of wall hydraulic conductivity is probably approximately 1×10^{-7} cm/sec as based on quality control testing (Section 2). This value yields a cutoff wall efficiency in excess of 99%.

- The overall passive containment efficiency, including bedrock leakage, is also important. The overall efficiency of the passive containment elements (cutoff wall, cap and fractured bedrock) was found to be approximately 55% (under worst case precipitation induced stress conditions) with the major loss flowing out through the bedrock aquifer. It was recognized during the RI/FS phases of the project that the glacial till/bedrock aquitard/aquifer containment bottom would leak. However, additional passive barriers such as grouting to stop such leakage were not found to be economically feasible. Hydrodynamic isolation systems were therefore incorporated in the overall containment design. Hydrodynamic isolation not only proved more cost efficient than additional physical barriers, such as grouting the bedrock, but also provided for recirculation/treatment capabilities. Following this approach, a purge stream pumpage was instituted in 1986 when the groundwater treatment system went on line. At this time, over 72,000 GPD are being discharged outside the cutoff wall in order to implement the hydrodynamic system elements.

- The contaminant migration analysis supported the overall conclusions derived form the hydraulic stress testing. The data indicated that contaminant flux, in the absence of hydrodynamic isolation, would be through the bedrock below the wall. However, the contaminant data was inconclusive when analyzed independently. This stems from the lack of sufficient data due to cost and conflicting objectives governing sample selection as well as delays in the start-up of the hydrodynamic isolation system due to construction contract difficulties.

The data obtained and the computations executed during this study in combination with long-term leachate/backfill compatibility testing undertaken as part of the cutoff wall design process indicate that significant chemical degradation of the cutoff wall is unlikely over the 2.5 years since its construction. The two pore volume exchanges required for chemical degradation are computed to take over 20 years under worst case conditions. Leachate/backfill compatibility testing indicates that the worst case leachate found at the Gilson Road site only increases the hydraulic conductivity of the intact backfill by a factor of two.

Design of Soil-Bentonite Backfill Mix for the First Environmental Protection Agency Superfund Cutoff Wall

by Donald Schulze, Matthew Barvenik
and John Ayres

Introduction

The effectiveness of soil-bentonite backfilled cutoff walls is dependent on several factors, the combination of which control the overall ability of this means of containment to significantly reduce the discharge of contaminated leachates from a given site. Some of these factors are site-specific and are subject to the variability of the geologic environment and the characteristics of the leachate discharge. Some are dependent on construction-related variables such as sloughing of in situ material from the trench walls into the backfill, inclusion of slurry-filled "windows" during backfill placement and/or the integrity of the bottom key. Other variables include the composition and properties of the proposed soil-bentonite backfill mix.

Procedures and findings described herein deal primarily with issues related to the latter; specifically, the hydraulic conductivity of the backfill mix and a quantitative assessment of the susceptibility of the mix to chemically or physically degrade after permeation by contaminated leachates. The work acknowledges the fact that an engineer should be able to adequately predict both the short-term and long-term behavior of backfill materials that he is designing. However, restraints imposed by time and money are also recognized. The resulting end point, the design backfill mix, is arrived at through a series of iterative approximations and should, therefore, be tempered with appropriate engineering factors of safety.

Background

The Gilson Road uncontrolled hazardous waste disposal site was the subject of the first cooperative agreement signed under EPA's Superfund program. Clandestine dumping of toxic, organic chemicals into the soil and aquifers underlying the properties resulted in a contaminated plume more than 450m (1,500 feet) in length, up to 33m (110 feet) in depth and covering about 120,000m^2 (30 acres). Discharge of the pollutants into local streams not only presented a health hazard to nearby residents, but also threatened downstream municipal drinking water supplies.

The site is located in a suburban area in Nashua, New Hampshire, and is surrounded by homes and trailer parks. Disposal of drums and chemical sludges took place simultaneously with landfilling operations in an abandoned 24,000m^2 (6 acre) sand and gravel borrow pit. In addition, more than 4,000,000 L (1,000,000 gallons) of liquid chemical waste were discharged directly to a subsurface leaching area adjacent to the borrow pit. State regulatory officials implemented legal actions to stop the disposal and issued contracts for drum removal and investigative studies in mid 1980. Investigations took place over a period of approximately one year and identified a stratified plume containing volatile organic solvents at levels exceeding 2,000 ppm. The hydrogeologic analysis performed on site indicated that most of the contaminants were moving through pervious sands and gravels at a rate of about 7×10^{-4} cm/sec (2 feet/day).

Investigative studies resulted in a report that was submitted in July 1981. Cost approximations were presented for a variety of interim and final remedial measures. These included several combined hydrologic-isolation and ground water treatment scenarios, as well as total removal alternatives. Based on the high costs associated with total removal alternatives, a containment using a soil-bentonite backfilled cutoff wall as well as ground water recovery/treatment was recommended. Modifications to the original plan and the need for more time to complete additional impact evaluations and prepare final design criteria, delayed the implementation of the scheme. It was recognized that such delays would result in a significant discharge of highly contaminated pollutants to the stream and further degradation to both air and water quality in and around the site. Interim emergency action was undertaken through the U.S. EPA for design, construction and operation of a temporary ground water recirculation system. Ground water was extracted by pumping wells located near the downgradient edge of the highly contaminated zone and discharged, untreated, to a shallow trench located about 125m (400 feet) upgradient. The sole purpose of the system was to temporarily retard the movement and discharge of the plume until completion of the cutoff wall in November 1982.

Final construction involved containment and capping of 80.000m^2 (20 acres), including about 20,000m^2 (204,000 feet^2) of cutoff wall ranging in depth from 10 to 33m (30 to 110 feet). The configuration of the wall and capped areas is shown in Figure 1, as is the location of the temporary ground water recirculation system. Design of a ground water recovery and treatment plant has been completed and construction of this facility is now underway.

Laboratory Testing Programs

Three laboratory testing programs were proposed and implemented in 1981 during design and construction of the Gilson Road cutoff wall. The first was, of necessity, limited in duration and provided data on which the design of the wall backfill mix was based. Laboratory testing programs utilized during design are the subject of this paper and are described later. A second program involved long-term hydraulic conductivity testing of the design backfill mix; these tests were initiated prior to construction and have continued over the past two years. Findings of the long-term hydraulic conductivity testing are now being evaluated and shall be the subject of a future paper. The third program was completed during construction of the wall as a portion of the quality control program. Results of the construction control testing have been described previously (Ayres et al. 1983). These data and procedures are now being evaluated for incorporation as standard guidelines for quality control testing of soil-bentonite backfills being prepared under a separate EPA research contract.

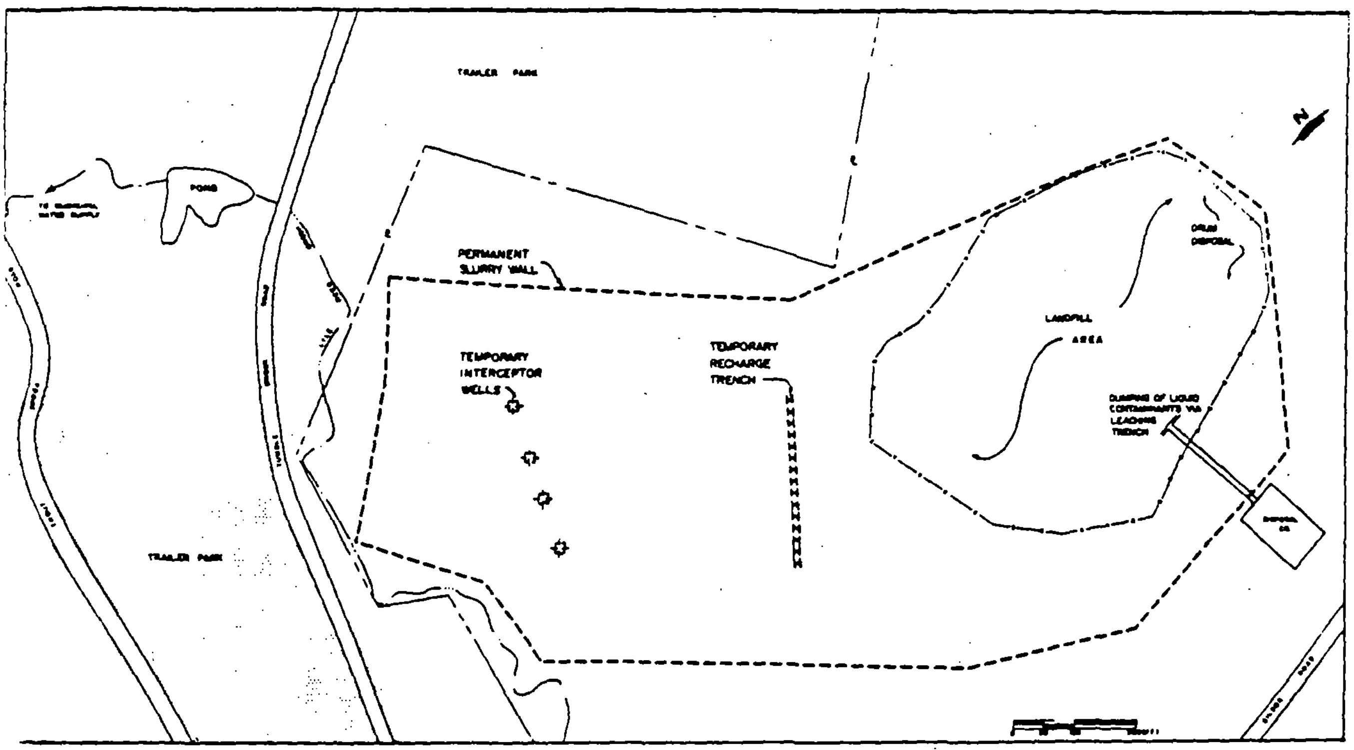
In the most simple of terms, the primary purpose for testing potential backfill materials during design of a soil-bentonite cutoff is to identify a cost-effective mix that will meet or exceed the specified hydraulic conductivity requirements for the completed wall. These requirements are typically expressed in terms of performance; for instance, "the gradation and materials used for backfill shall be such that the slurry wall barrier achieves an effective, long-term, hydraulic conductivity of less than 1×10^{-7} cm/sec with site leachate as the permeant" (Ayres et al. 1983). As such, the testing program implemented during design should:

- Establish a range of cost-effective backfill mix gradations
- Assess the "short-term" hydraulic conductivity of the backfill mix under "worst-case" conditions of mixing, stress, gradient and temperature expected in the field
- Evaluate the "long-term" change in hydraulic conductivity expected due to degradation by site leachate.

The steps involved in this design process are presented as a flow chart in Figure 2. The procedures outlined in the flow chart are discussed in more detail after a summary of general testing considerations required to ensure the validity of the results obtained. Requirements for long-term testing are also shown in the flow chart.

Hydraulic Conductivity Tests, Physical Parameters

Design backfill mixes incorporating various mixtures of on-site soils, off-site borrow and bentonite are typically evaluated using hydraulic conductivity determinations. Efforts must be made to provide laboratory simulations that will approximate actual "field conditions." The most significant of these physical parameters are the manner in which the backfills are mixed at the



gure 1. Site plan

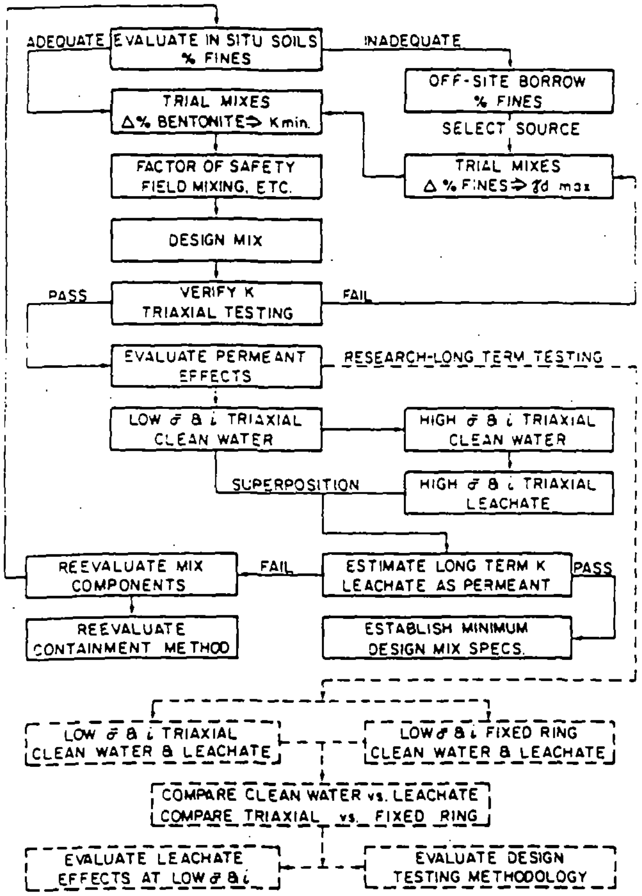


Figure 2. Backfill mix design procedure

trench as well as the state of stress, gradient and temperature that would exist throughout the wall after construction has been completed. These concerns are treated in the following subsections.

Mixing

Mixing of backfills usually takes place at the edge of the trench, where on-site soils may be combined with off-site borrow materials to which some percentage of dry bentonite is added. This mixture is then wetted to a specified slump (for instance 10 to 15cm (4 to 6 inches)) by sluicing with bentonite slurry. The actual mixing is usually accomplished by repeatedly tracking a bulldozer through the materials. Such procedures are crude at best and are likely to result in imperfect blending of the mass mix, particularly with respect to dispersion of the bentonite powder. For this reason, hydraulic conductivity values established from controlled laboratory mixes should be considered as lower bounds leading to specified bentonite quantities and/or mixes, which are more conservative.

Stress

The state of stress, σ, in a completed soil-bentonite wall increases from the surface of the wall to the bottom key. This stress increase is equivalent to the effective unit weight of the backfill mix per foot of depth. This assumes that no arching of the backfill occurs, whereby stresses are not transmitted throughout the depth of the wall. An increase in stress leads to consolidation of the backfill, which will decrease its void ratio. This decrease, in turn, results in a lower hydraulic conductivity. Therefore, assuming that the backfill is a homogenous mass, the hydraulic conductivity should decrease with depth in the completed cutoff wall.

Inasmuch as the design mix and specified upper-bound hydraulic conductivity pertains to the entire depth of wall, stresses applied to a laboratory specimen during hydraulic conductivity testing should be those in which the void ratio is similar to that expected for the worst-case field situation. This is typically considered to occur in the upper 10 feet or "top of wall." Based on this rationale, if the specified hydraulic conductivity value is met for top-of-wall conditions, the remaining portions should also meet the criteria. Conversely, a backfill mix designed from tests performed at stresses significantly higher than those equivalent to the top of wall is unconservative.

Gradient

The hydraulic gradient, *i*, in a completed homogeneous wall keyed to a relatively impervious base, is essentially constant with depth for any given point along the perimeter. By predicting ground water levels inside and outside of the containment, the gradient across the wall is simply calculated as the head loss per unit width of wall. During laboratory hydraulic conductivity testing, however, a gradient of any value may be simulated by applying differential pressures across the specimen. Although hydraulic conductivity is relatively insensitive *to gradient at a given void radio,* increased "confining stress" (decreased void ratio) as the gradient is increased is inherent in the testing equipment. This is true for both triaxial testing, via differential pressure across the membrane and fixed ring testing via head loss within the sample itself. Hence, in an effort to rapidly permeate a required number of pore volumes of leachate through a sample, the gradient may be established at a high value as compared to the field situation. An unrealistically high gradient, through its effect on stress and void ratio, may result in an artificially low (unconservative) estimation of hydraulic conductivity for the backfill at the top of wall as discussed above.

Temperature

An additional factor requiring laboratory control is temperature. The temperature of the permeant governs its viscosity, which in turn, affects hydraulic conductivity. In most instances, a decreased temperature produces an increased viscosity, which results in a lower hydraulic conductivity. Therefore, the temperature at which the test is run should simulate expected in situ temperatures in the field. In addition, the rate of chemical reaction between the permeant and certain backfill constituents may double for every 10 C increase in temperature (Strum and Morgan 1981). Thus, a chemical reaction with the backfill mix may be unrealistically accelerated if laboratory temperatures exceed those expected in situ.

Hydraulic [redacted]

Degradation, by way of contact and subsequent permeation of a backfill mix with certain leachates, should be recognized as a condition, which may alter (increase) the hydraulic conductivity of the in-place wall. Specifically, [redacted]

• Structural damage of the backfill materials, i.e. strong organic and inorganic acids and bases may dissolve or alter the bentonite portion of the backfill or, in some cases, the soil portion itself. This may lead to a large increase in hydraulic conductivity.

• Depression of the double layer around the bentonite clay particle, i.e. a decrease in thickness of the bound ion-water layer around the clay particle. The resulting smaller "effective clay particle size" may lead to an increase in the hydraulic conductivity of the *intact* backfill and/or may cause the backfill to shrink and crack depending on the state of stress existing in the wall.

A knowledge of the tendency for certain chemical onstituents to alter the bentonite clay particle is, at this oint, far from complete. Certain compounds in conentrated form are known to be desiccants and will ffect the double layer. Acetone is an example. At what oncentrations this may happen, whether or not the resence of other constituents will accelerate or attenue the reaction, and, in general, which of a potentially rge number of inorganic and organic compounds ould cause alteration or degradation of the mineral ructure, are questions without answers.

Chemical analysis of the site leachate should be ndertaken prior to design of the backfill mix. However, nless there is information available that will describe e effects on the backfill of the specific concentrations d proportions of compounds identified, hydraulic nductivity testing of the proposed backfill should be rformed using the worst-case site leachate as a pereant. Efforts must be made to preserve the chemical egrity of the leachate at in situ conditions during the draulic conductivity testing. Ideally, during leachate rmeation, backfill samples should also be subjected worst-case physical conditions projected for the ual wall. These conditions include state of stress, dient and temperature as previously discussed. Unfornately, as of this writing, available "protocols" indicate t permeation of at least two pore volumes is required assess chemical degradation. If worst-case (top-ofll) stresses are simulated, the maximum gradient and is rate of permeation, is severely limited. Hence, the ration of testing to evaluate relatively impermeable kfill mixes (1×10^{-7} cm/sec or less) may exceed many nths and in some cases years. A design phase labora testing program intended to simultaneously model relative site conditions is, therefore, idealistic and kely to occur. Rather, a rapid testing methodology is uired during the design phase that would allow roximation of long-term backfill chemical behavior ler field conditions. The procedures presented herein utilize multiple tests to independently evaluate the influence of permeant, stress and backfill gradation on long-term hydraulic conductivity.

Hydraulic Conductivity Tests, Testing Equipment and Procedures

Considering the number of variables that the laboratory tests are attempting to model and the recognized limitations of the program, efforts should be made to use existing test equipment and procedures to the extent possible. Hydraulic conductivity testing may be done in a number of ways, each of which has its own advantages and disadvantages. Two traditional, yet contrasting, methods for determining hydraulic conductivity make use of either a flexible membrane or a rigid or fixed-ring confining media.

Flexible Membrane Equipment

Testing procedures that employ a flexible rubber membrane to confine the sample usually make use of standard soil-testing triaxial devices (Figure 3). The prepared soil sample resting on a pedestal and porous stone, is encased in a flexible membrane, fitted with a top cap and sealed top and bottom using rubber "O" rings. After sample preparation, the cell is filled with a fluid (generally water) and pressure is applied both within (as back pressure) and around the sample (cell pressure) to simulate expected in situ stress conditions. A permeant may be introduced into the sample and monitored vs. time, yielding the value of interest (i.e. hydraulic conductivity). Additional triaxial equipment design details can be found in Bishop and Henkel (1962).

The advantages of this means of testing are:

• Complete saturation of the test sample may be obtained and verified prior to determining the hydraulic conductivity by applying additional back pressure

• The flow along the sample-membrane boundary is negligible as the confining stress presses the flexible membrane against the sample, regardless of irregularities.

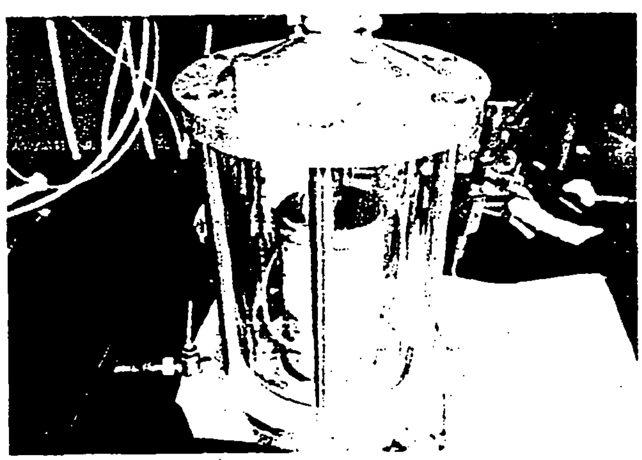


Figure 3. Standard triaxial testing device

The disadvantages of these procedures are:

• The initial cost of purchasing triaxial equipment is substantial as are replacement costs found to be necessary if caustic or acidic leachates are used as permeants

• Sample preparation is often difficult and time-consuming, especially for those samples that are prepared at low densities equivalent to top-of-wall backfill conditions

• If the soil-bentonite mix desiccates and shrinks during leachate permeation, the flexible membrane will follow radial and axial deformations, thus maintaining an "intact" sample. This may or may not be a disadvantage depending on the state of stress in the actual field case. However, it is emphasized that this testing method will result in unconservative values of hydraulic conductivity if the backfill exhibits a tendency to shrink and does not behave plastically under field stresses (cracks).

Fixed Ring Equipment

Procedures that use rigid walls or fixed rings to confine a test sample have been in use for many years. They may employ undisturbed sampling tubes, API filter cells, consolidometers, compaction molds and other devices. The API cell (Figure 4) is an inexpensive 3-inch diameter, fixed-ring permeameter adopted by the American Petroleum Institute for testing filtrate loss of bentonite slurries. With minor modifications, the apparatus may be used to permeate a soil-bentonite backfill sample under stresses similar to actual field conditions. Alternatively, thick walled tubes of any diameter may be fitted with specially fabricated end caps and "O" ring seals to allow pressurization of the system (Figures 5 and 6).

The advantages of fixed-ring testing procedures are:

• The initial and replacement costs are significantly less than those of a triaxial system

• The time involved in setting up a sample and performing a test is minimal.

• If the soil-bentonite backfill shrinks due to desiccation during leachate permeation, the rigid wall remains fixed and the sample cracks or separates from the cell wall leading to high values of hydraulic conductivity. This may or may not be an advantage depending on the state of stress in the actual field case. Although this test procedure should always yield a conservative estimate for hydraulic conductivity, the values obtained may be so overconservative as to preclude the use of the backfill mix when, under actual field stresses, the sample may behave plastically in response to shrinkage (no cracking).

The disadvantages of fixed-ring testing methods are:

• Flow along the boundary of the backfill-rigid wall interface may be significant, resulting in hydraulic conductivity values that are artificially high

• The degree of saturation of the sample cannot be verified prior to testing. Therefore, the hydraulic conductivity value may not be representative of a saturated condition and thus be artificially low.

Procedures

In performing triaxial hydraulic conductivity tests, no modifications to the standard apparatus are needed. However, the volume of water in and volume of water out of the sample should be measured independently. This allows for determination of the end of consolidation after application of the confining stress, the end of swelling due to the reduction in effective stress upon application of the driving head (increase in pore pressure with no change in total stress) and the integrity of the system with respect to leakage.

During preparation for triaxial testing, the sample must not slump on the cell pedestal after the former is

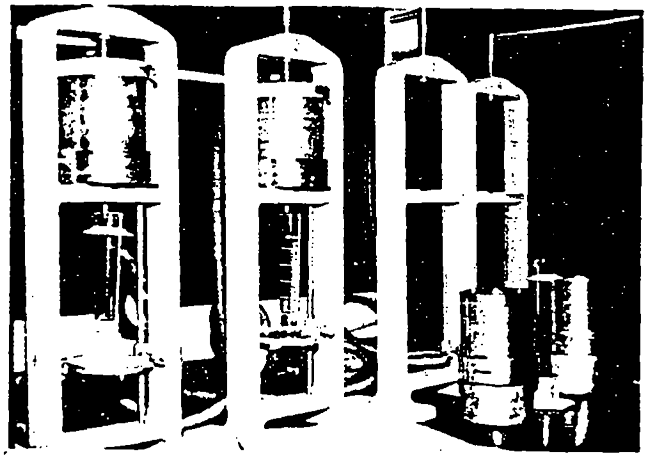


Figure 4. API fixed-ring cells

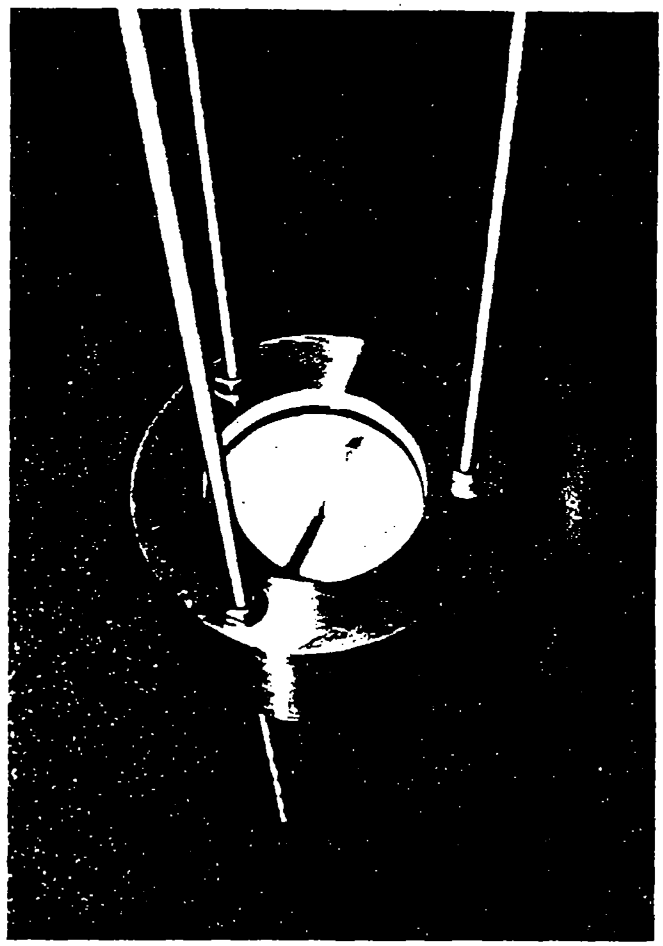


Figure 5. Fixed ring end caps with 'O' ring seals

removed. This causes the sample to severely distort from a right circular cylinder. The conventional means of applying a vacuum or negative stress to the sample is unacceptable because of the impermeable nature of the mix. In order to prevent slumping, a piece of aluminum foil is placed around the circumference of the former, outside the flexible membrane. The foil is perforated allowing the membrane to be drawn tight against the former with a small vacuum. Upon removal of the former, the aluminum foil surrounding the membrane is rigid enough to support the sample while the triaxial cell is filled and a confining pressure applied. The remainder of the test is performed in accordance with procedures outlined in the U.S. Army Corps of Engineers Manual (1970). It is important to note that the aluminum foil surrounding the sample should be visibly "crinkled" after the sample is consolidated and backpressured, indicating that the membrane and foil continue to act as flexible confining medium.

In the case of fixed-ring, "quick" test procedures, the API filter cell is particularly useful. However, the following modifications should be made to the device if purchased "off the shelf."

- The accompanying pressure system should be modified to accept a regulator for each cell along with a gauge that has a range of 1 to 15 psi for application of low pressures
- Bentonite "paste" should be applied to the inside diameter of the rigid wall and trimmed to a uniform thickness of 1/32 inch using a trimming jig. This will limit boundary flow along the backfill-rigid wall interface. Other agents such as special silicone greases may also be used for this purpose. The material selected must conform to the sample and exhibit a hydraulic conductivity of at least one order of magnitude less than that of the backfill.
- Test samples should be fabricated on a thin bed of Ottawa sand underlain by a porous filter pad to prevent plugging of the bottom outlet port.

Gilson Road Site Hydraulic Conductivity Test Program

As indicated in previous sections, testing that simultaneously approximates field stress conditions and assesses chemical effects after a two to three pore volume leachate displacement is the most accurate way to evaluate a backfill design mix. This type of testing is inherently a long-term endeavor and, as such, cannot be completed during the design phase of a project. Therefore, procedures used to arrive at a backfill gradation during design must rely on superposition of results of individual sets of tests. In general, the methodology involves:

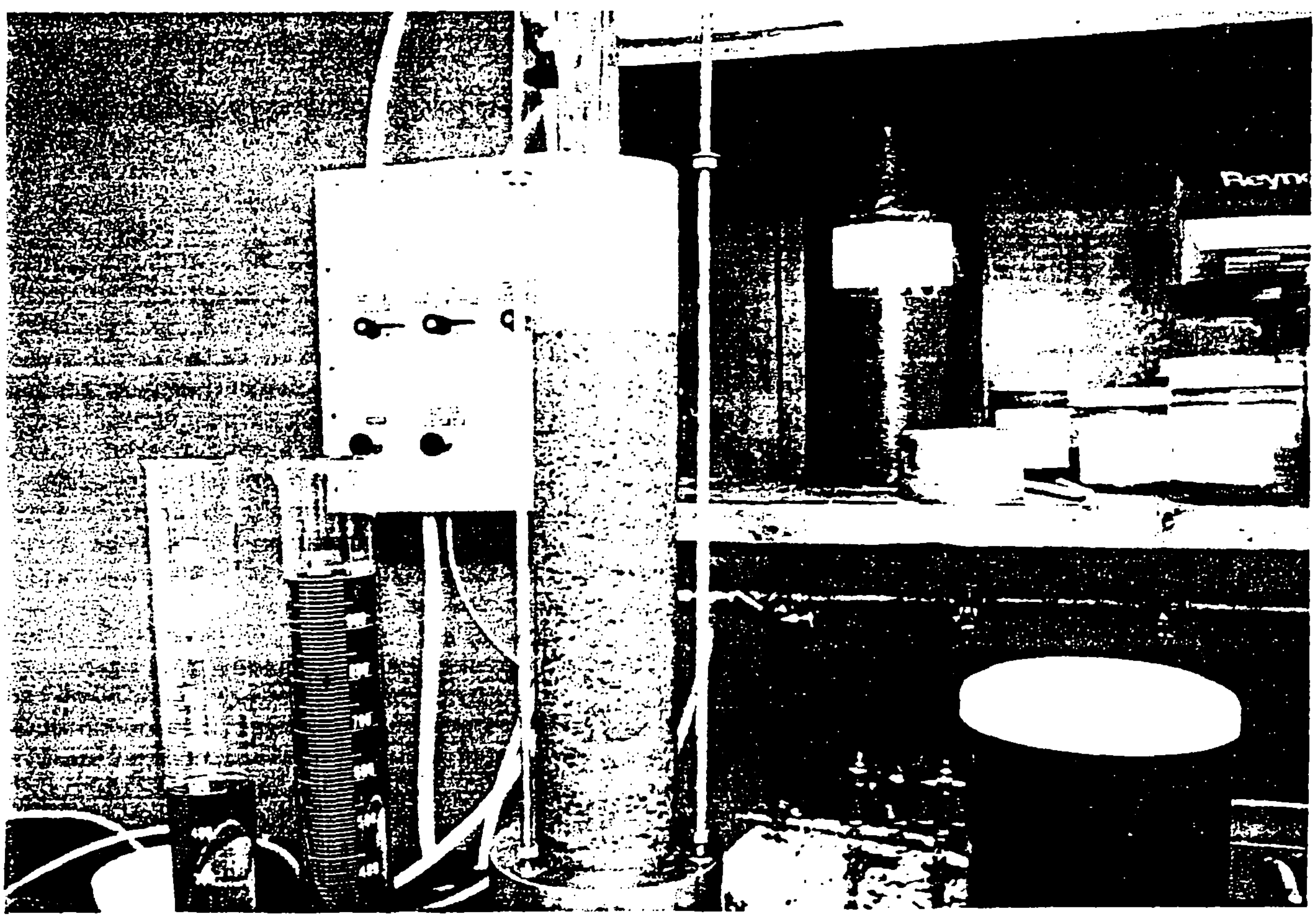


Figure 6. Plexiglass fixed ring

• Selecting appropriate gradations for the design backfill mix based on cost and hydraulic conductivity criteria

• Establishing the hydraulic conductivity of the proposed backfill mix under the worst-case field state of stress (top of wall) using clean water as the permeant

• Evaluating the chemical effect of the site leachate on the backfill using accelerated permeation rates. The accelerated permeation requires high gradients and, therefore, confining stresses well above those representative of top-of-wall conditions.

• Applying the percent change in hydraulic conductivity determined under leachate permeation to the results of the clean water tests, which simulated worst-case stresses.

• Compare the resulting estimated long-term hydraulic conductivity to the performance specification.

This procedure is subsequently presented in more detail by utilizing the Gilson Road project as an example.

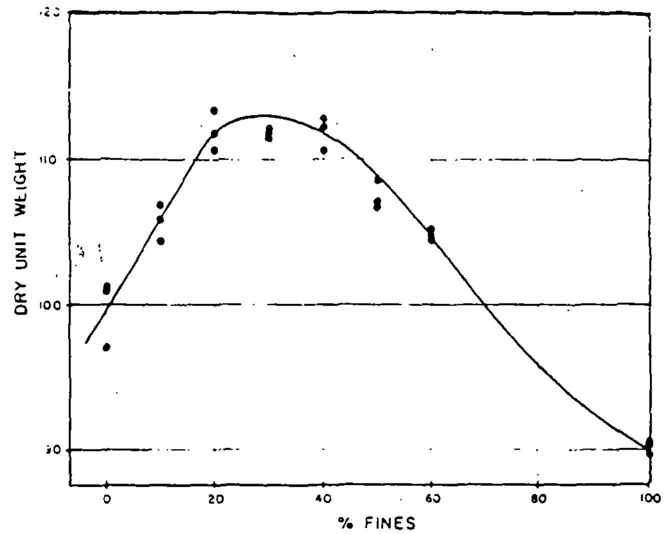


Figure 7. Percent fines selection

Evaluation of In Situ Soils

The initial portion of the laboratory testing program centers around evaluation of the in situ soils at the site as to suitability for incorporation as part of the backfill mix. Where on-site soils consist of coarse-grained granular deposits only, either large quantities of bentonite, or some proportion of finer-grained soils from off-site sources together with a smaller amount of bentonite, would be required. Owing to the relatively high cost of bentonite and the possibility of chemical degradation, the latter option is usually more cost-effective and technically desirable. It should also be recognized that variable soil conditions may exist on any given project site and that several in situ soils and potential off-site borrow gradations may need to be evaluated. Therefore, the initial testing must be designed to quickly assess the various combinations that could result in an acceptable backfill mix.

At the Gilson Road site, the on-site soils were typically coarse sands and gravels containing less than 5 percent fines with a resulting average hydraulic conductivity of about $4x10^{-2}$ cm/sec. This was over four orders of magnitude greater than the performance criteria specified for the soil-bentonite backfill ($<1x10^{-7}$ cm/sec). Although the hydraulic conductivity could be reduced by adding bentonite alone, laboratory testing indicated that as much as 10 percent by dry weight would be required. Therefore, material from off-site locations that contained high proportions of "fines" was evaluated for use in blending with the indigenous soils in order to reduce the amount of bentonite needed.

Several borrow sources were examined that were within 10 miles of the site. Representative samples of materials present at these sources were subjected to a series of simple soil tests, namely, sieve analyses for the percent fines (- #200 sieve sizes) and hydrometer analyses of these fractions for clay content. Results of these tests were used to narrow the number of sources and materials on which additional testing was performed. The borrow ultimately selected for use at the Gilson Road project consisted of a very fine sandy silt containing 50 to 70 percent nonplastic fines. This material was added to and mixed with the in situ material excavated from the trench at a rate of approximately 50 percent.

Determination of Percent Fines

An initial series of eight backfill mixes was prepared by blending off-site fines from the previously determined borrow source with on-site soils in such a manner as to achieve 0, 10, 20, 30, 40, 50, 60 and 100 percent fines with no bentonite added. These mixes were saturated with water and placed in constant volume molds at near minimum density. The low density was required in light of the probable density characteristics of the final backfill mix at the specified 10 to 15cm (4 to 6 inch) slump.

The procedures used were similar to standard geotechnical proctor testing except that all the samples were saturated and a very low compaction effort was used. The objective of this testing was to determine the amount of fines required to just fill the voids between the larger soil particles representative of the in situ soils on site. This point is indicated by a maximum unit weight. As seen in Figure 7, the unit weight peaked with the addition of between 20 and 40 percent fines and then decreased with higher proportions of fines. These data follow trends expected based on soil mechanics theory. The point of maximum density corresponds to minimum void ratio and, thus, should yield a minimum hydraulic conductivity for the materials being evaluated. A series of API hydraulic conductivity tests were then performed on a mixture of 70 percent in situ soils and 30 percent fines. The tests yielded an average hydraulic conductivity of about $2x10^{-5}$ cm/sec. This mix was subjected to further augmentation with bentonite as described subsequently.

Determination of Percent Bentonite

The mix containing 30 percent fines was split into four aliquots, and 0, 2, 4 and 6 percent dry bentonite by weight was added to the dry soil. These samples were sluiced with a previously hydrated six percent bentonite slurry to a slump of 14 to 15 cm (5½ to 6 inches). The addition of the slurry increased the total bentonite content to 1.1, 3.6, 7.0 and 10.0 percent, respectively. Three splits of each of these four mixes were set up at their slump densities in the API apparatus. A driving pressure of 21 kilopascal (3 psi) was applied across the sample. Hydraulic conductivities were obtained within 24 hours after the sample had consolidated and the readings stabilized. No attempt was made to saturate the specimens. These tests (Figure 8) indicated that between 1.5 and 3.5 percent total bentonite and 30 percent nonbentonite fines would yield the mix with the lowest hydraulic conductivity; about $2x10^{-8}$ cm/sec *under laboratory mixing conditions*. As can also be seen in Figure 8, additional bentonite would actually increase the hydraulic conductivity slightly. This result, although initially surprising, follows theoretical trends. Further addition of bentonite past the point of filling voids in the granular soils yields a significant increase in water content and decrease in unit weight as shown in Figure 9. This should correspond to an increase in void ratio and thus hydraulic conductivity.

To account for imperfect mixing in the field, an additional two percent bentonite was specified. A minimum design criteria was thus established requiring a total of five percent bentonite in the backfill mix. A second series of hydraulic conductivity tests was performed using the API cells on this mix with 30 percent fines and five percent bentonite. Results ranged from $2x10^{-8}$ cm/sec to $4x10^{-8}$ cm/sec. During construction, more than 85 samples of the backfill were taken and tested for percent bentonite and hydraulic conductivity. These data (Ayres et al. 1983) indicated that an average of four percent bentonite was actually achieved in the field, resulting in an average hydraulic conductivity of about $5x10^{-8}$ cm/sec.

Determination of Hydraulic Conductivity, Low Stress

In order to verify the API results, similar samples (30 percent fines and five percent bentonite) were set up in triaxial cells and permeated with clean water. The density was essentially identical to that used in the API tests. Samples were consolidated to an effective stress of 21 kilopascal (3 psi), backpressured to saturation and permeated under a head equivalent to about 100cm (3.5 feet) of water. The permeability value reported was an average number obtained after the sample had stabilized with respect to the applied stress. These tests were "short term," i.e. the hydraulic conductivity was defined after several consistent values were obtained (generally two to three days) rather than after a specific pore volume displacement. The values of hydraulic conductivity obtained from the triaxial tests agreed well with the API tests. Additional comparisons between API and triaxial results were performed during construction. These data (Ayres et al. 1983) indicate a "one-to-one" correlation within a one-fourth order of magnitude error band.

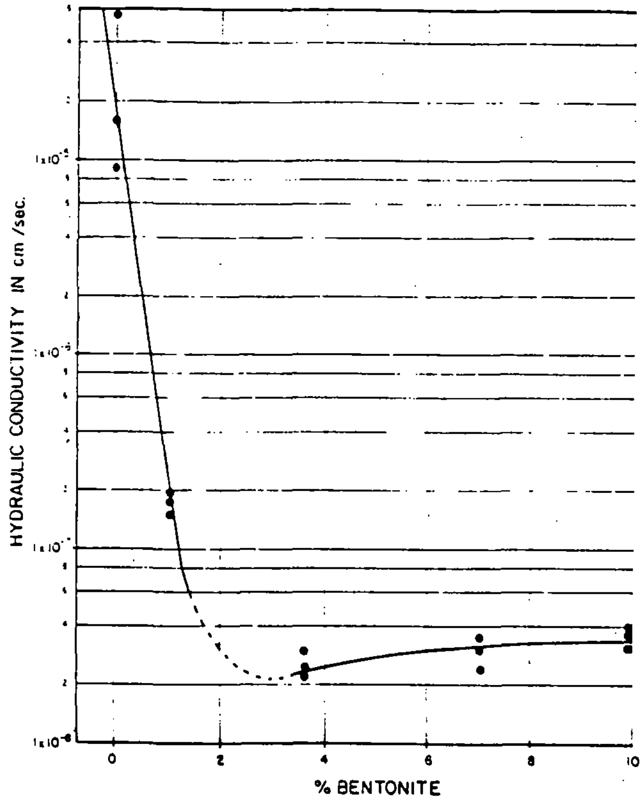


Figure 8. Percent bentonite selection

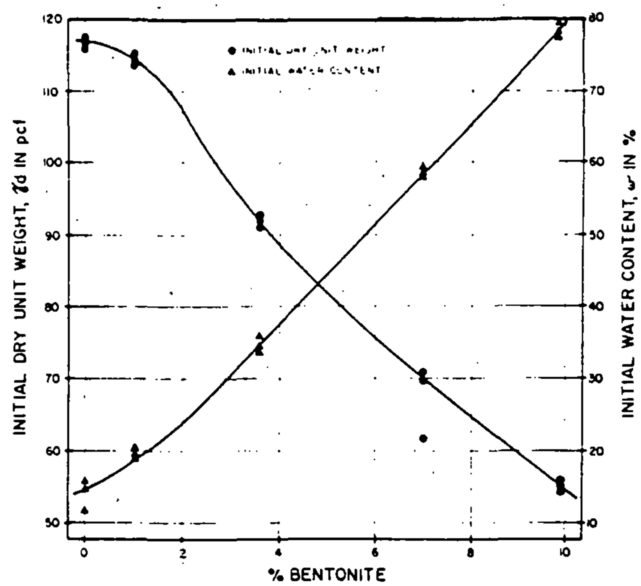


Figure 9. Backfill characteristics vs. percent bentonite

At the conclusion of this portion of the testing program, tentative design criteria were established. The resulting specifications required "*not less than* 30 percent fines and five percent bentonite" (Ayres et al. 1983). Figure 10 shows the gradation characteristics of the on-site soils, off-site borrow and the tested design mix, which included five percent bentonite.

[redacted]

The site leachate contained relatively high propor-

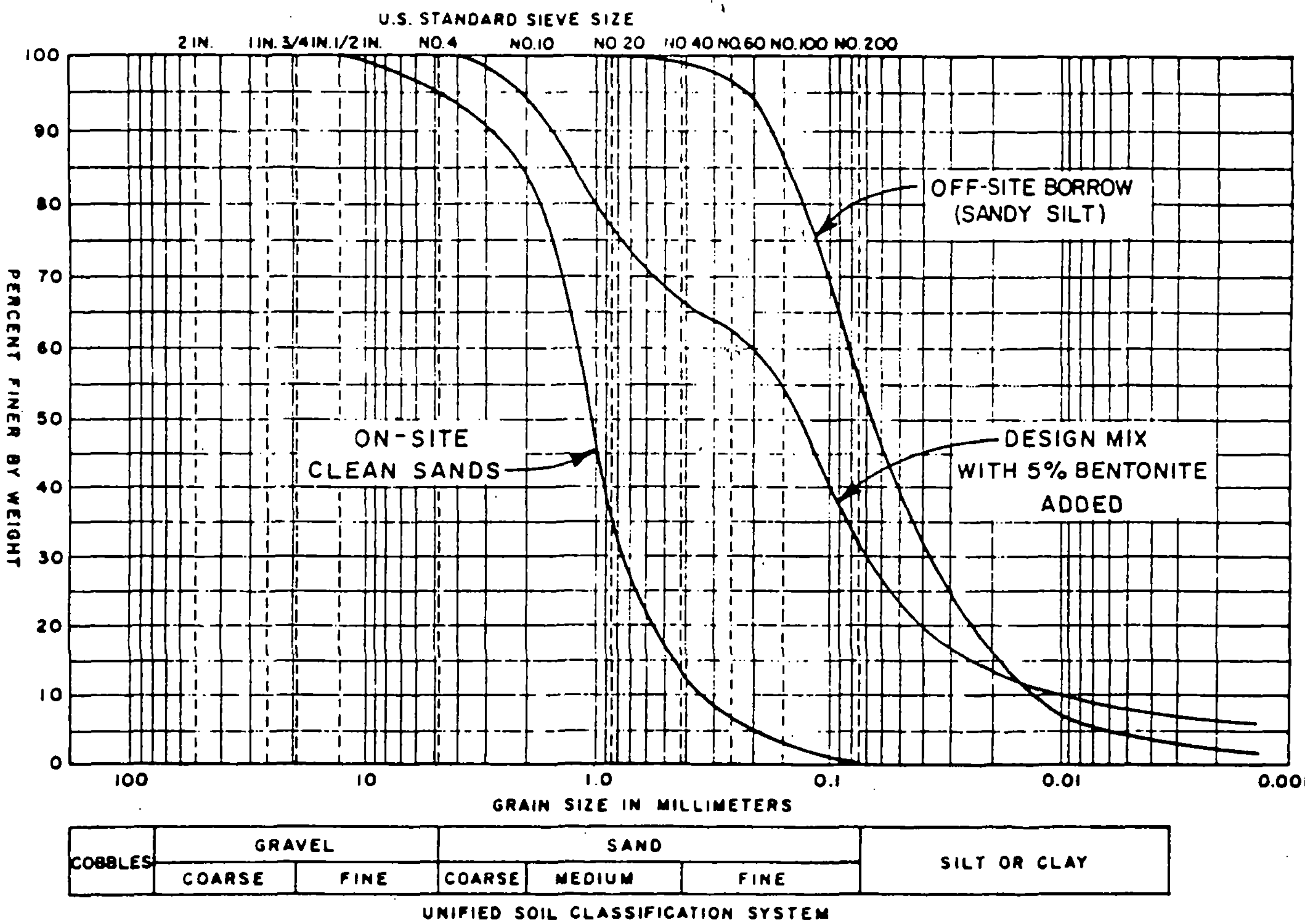


Figure 10. Gradation curves

tions of volatile and extractable organic chemicals, some of which, at high concentrations, were known to potentially degrade or alter bentonite. Estimates based on assumed field gradients, a laboratory simulated void ratio and a hydraulic conductivity of 1×10^{-7} cm/sec (as specified) indicated that displacement of two to three pore volumes of leachate through the wall would be expected in about 10 to 15 years. Should degradation of the bentonite occur during this period, the escape of contaminants through the containment would increase. Therefore, during the testing program, attempts were made to quantitatively assess the potential effects of the leachate on the design backfill mix. As stated previously, time limitations precluded the use of laboratory testing that would realistically model all expected field conditions.

The program implemented was as follows:

- The triaxial samples tested previously (five percent bentonite/30 percent fines) were further consolidated and permeated with clean water under a high gradient; hydraulic conductivity values were determined. Although the high effective consolidation stress caused a large reduction in hydraulic conductivity, the proportionately greater increase in gradient permitted faster pore volume exchange.
- Site leachate was then introduced as the permeant and the change in hydraulic conductivity recorded after an exchange of one to two pore volumes. This testing required between 60 and 90 days.

The results of the tests using clean water as a permeant were compared to those after the permeant was changed to leachate. Changes in hydraulic conductivity were noted, increasing two- to three-fold during the tests. This ratio, when applied to values obtained from tests run at low stresses with clean water, yielded values within the specified limit ($<1\times10^{-7}$ cm/sec).

Acknowledging that definition of failure under the previously described procedures was subjective, long-term hydraulic conductivity tests were recommended in order to further evaluate the potential for eventual degradation of the backfills under more realistic testing conditions. Such testing would permit a more objective assessment of the quantity of leachate expected to enter the environment in future years. The results of these tests, wherein confining media, stress conditions, temperature and permeant simulated worst-case field conditions, are now being evaluated.

Conclusions

Laboratory testing programs utilized in the design of soil-bentonite backfill mixes require careful assessment of the physical conditions expected to exist in the completed cutoff wall as well as the chemical characteristics of the site leachate. Additional variables inherent in the procedures and materials used to construct the containment must also be accounted for. Testing can be performed that simultaneously models low stresses and gradient, temperature and the chemical nature of the permeant. However, the long testing periods needed to displace the required number of pore volumes for assessment of chemical degradation preclude the use of this type of testing during the design phase. A relatively quick testing program was therefore developed in 1981 to establish minimum criteria for the soil-bentonite backfill mixture specified on the Gilson Road project. This procedure utilized a large number of quick and inexpensive "API" (fixed ring) hydraulic conductivity and unit weight tests to establish preliminary mix proportions. Two independent series of triaxial (flexible membrane) tests were then performed to determine separately the effects of low stress/gradient and chemical degradation on hydraulic conductivity. The results were superimposed to evaluate the projected long-term performance of the backfill mix. Based on the testing and design protocol presented, the very coarse and permeable nature of the in situ soils at the Gilson Road site required the addition of off-site fines in order to cost-effectively achieve the specified hydraulic conductivity (not greater than $1x10^{-7}$ cm/sec). The final design mix specification required not less than 30 percent fines and five percent bentonite.

It must be emphasized that although the testing procedure presented herein is still routinely being employed in the design of soil-bentonite cutoff walls, it relies heavily on triaxial procedures for predicting the effect of chemical degradation. *Initial* results from the long-term *triaxial* tests initiated prior to construction (simultaneously simulate field stress and permeant) indicate that the procedure adequately predicts the long-term change in hydraulic conductivity due to chemical degradation. However, preliminary review of the long-term *fixed ring* test data indicates much greater increases in hydraulic conductivity due to chemical permeation. Comparison of the long-term fixed ring and triaxial data appear to indicate that not only does the hydraulic conductivity of the intact sample (triaxial) increase with chemical permeation, but the backfill undergoes volumetric shrinkage due to desiccation. Hence, the fixed ring confining apparatus predicts catastrophic failure due to "cracking" of the sample.

It is emphasized that data from the long-term testing are preliminary in nature and have not fully been analyzed. These data and a complete analyses will be presented in a future paper. Although preliminary, the discrepancy between the long-term fixed ring and triaxial data indicates that caution must be used in the design of soil-bentonite walls if based solely on triaxial testing. Such caution should prevail until appropriate soil-structure interaction modeling has been completed to determine if the backfill behaves plastically (triaxial test) or rigidly (fixed-ring test) under the state of stress existing in the completed wall. In light of the above data and in recognition of the unknowns in any underground construction procedure, it is further recommended that hydrologic isolation be considered for use as a backup system where physical barriers are constructed to contain hazardous wastes.

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Biographical Sketches

As director of laboratory services for Goldberg-Zoino & Associates, Donald Schulze is responsible for coordinating and supervising all soil-testing activities with the firm's corporate headquarters and its satellite branches. He has a B.S. degree in civil engineering from Northeastern University and has been with the firm for more than 14 years, the last five of which have been as laboratory director. Schulze has supervised or participated in more than 3,000 laboratory testing projects including nuclear power plants, pumped storage hydro projects, highwall stability analysis for deep-coal strip mines, coal fly ash research programs, earthen and refuse dams, slurry wall backfill design and deep seabed projects for disposal of radioactive wastes.

With B.S. and M.S. degrees in engineering from Northeastern University and MIT, respectively, Matthew J. Barvenik has more than eight years of professional experience. As a senior engineer with Goldberg-Zoino & Associates, he has managed numerous instrumentation, geotechnical and geohydrological projects, including slurry wall supported tunnel excavations, hazardous waste investigations-containment, and custom laboratory and field instrumentation design. As principal with BarCad Systems Inc., he has consulted nationwide with respect to state-of-the-art sampling of contaminated soils and ground water. Barvenik's most recent work involves design of laboratory leachate permeation procedures, field quality control testing and field instrumentation for evaluation of soil-bentonite cutoff wall containment systems.

As chief geologist and a principal of the firm, John E. Ayres is responsible for managing geologic and hydrogeologic activities of Goldberg-Zoino & Associates. He is a certified professional geologist with more than 20 years experience. Ayres has served as principal-in-charge of many projects involving hazardous wastes, including 16 of the 38 New England sites listed as having Superfund priority. Ayres has also served as an expert witness in civil and criminal cases involving hazardous wastes and has given numerous presentations, lectures and papers on the subjects of ground water monitoring and containment/restoration at waste disposal sites.

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David M. Nielsen, Director of Research
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Soil-Bentonite Backfill Mix Design/Compatibility Testing: A Case History

Jane M. Bolton, E.I.T., David L. Jaros, P.E.,
Gordon G. Lewis, James M. Zeltinger, P.G.

U.S. Army Corps of Engineers
Omaha District
215 No. 17th Street
Omaha, Nebraska 68102
(402) 221-4169

INTRODUCTION

Soil-bentonite slurry trenches have been used in the U.S. as subsurface groundwater barriers since the 1940's (D'Appolonia, 1980). Construction consists of excavating the trench (typically 2-5 feet wide, keyed 3-5 feet into an impermeable formation such as rock or clay) while pumping in bentonite slurry to support the side walls. As slurry leaks into voids in the trench wall soils, clay particles build up in layers on the trench walls, forming a thin low permeability filter cake. The trench is then backfilled with a mixture of soil and bentonite, called the soil-bentonite backfill material. Backfilling with material of the proper consistency (unit weight about 15 pounds per cubic foot (pcf) greater than the slurry unit weight, with a concrete slump of 2 to 6 inches) does not substantially destroy the filter cake (D'Appolonia, 1980; Millet and Perez, 1981). Permeability of the completed trench is a function of both the filter cake and the soil-bentonite backfill material. The term "bentonite" is defined in the U.S. Environmental Protection Agency (USEPA) slurry trench design guidance document as a soil composed of at least 90 percent montmorillonite clay (JRB Associates, 1984). Many geotechnical textbooks, such as Lambe and Whitman (1969), define bentonite as montmorillonite clay containing primarily sodium as the exchangeable ions in its crystal structure. This paper utilizes the USEPA guidance document definition of bentonite.

The presence of chemical contaminants in soil and/or groundwater may significantly alter the rate of water movement through a soil-bentonite slurry trench (D'Appolonia, 1980; JRB Associates, 1984; Zappi et al., 1989b; Ayres et al., 1983). For example, calcium in soil or groundwater will displace some of the sodium ions in bentonite. This results in reduced swelling and increased permeability, not desirable for a groundwater barrier. While the effects of other individual chemicals have been studied and documented, the effect of multiple contaminants, which frequently exist at hazardous and toxic waste (HTW) sites, is largely unknown.

This paper presents a general overview of the Corps of Engineers Missouri River Division Laboratory (MRDL) mix design/compatibility testing methodology, while discussing in detail the testing program undertaken for the Lime Settling Basins (LSB) site at the Rocky Mountain Arsenal (RMA), Commerce City, Colorado. Objectives of the LSB testing program are to determine the optimum soil-bentonite backfill material mix design (soil and percent bentonite) necessary to achieve an in-place slurry trench permeability of 1 x 10-7 centimeters per second (cm/sec) or less, and to determine whether contaminants present in soil and groundwater at the LSB site will cause changes in soil-bentonite backfill permeability over time.

EPA /540/8-91/012

BACKGROUND

Site History. During the 1940's and 1950's, wastewater from production of Army agents was routinely treated prior to discharge to unlined evaporation ponds. This treatment involved the addition of lime to the wastewater to precipitate metals, principally mercury and arsenic. Wastewater produced in the South Plants was channeled into the LSB prior to gravity drainage to Basin A, an evaporation pond just to the north. The precipitation process produced a lime sludge that contained elevated levels of heavy metals, arsenic and mercury. Subsequent discharge of wastewater from production of pesticides resulted in the addition of pesticides to the LSB sludge. The LSB were removed from service in 1957. Studies have been conducted to characterize the nature and extent of contamination in the soil, sludge, and ground water in the vicinity of the LSB. The studies revealed the soil, sludge, and ground water contain elevated levels of organochlorine pesticides, organosulfer compounds, arsenic, mercury, and Inductively Coupled Plasma (ICP) metals (cadmium, chromium, copper, lead, and zinc).

Site Geology. Bedrock beneath the LSB area is the Cretaceous-Tertiary Denver Formation. The Denver Formation in the vicinity of the LSB consists of claystone and sandstone. The claystone is generally soft to moderately hard, brown to gray, and is occasionally silty. A thick, fine-grained sandstone lense is present in the northern section of the LSB area. The Denver Formation bedrock lies at depths of 13.0 to 33.0 feet below the surface in the LSB area. The local slope of the bedrock subcrop is about two degrees to the north-northeast. The dip of the Denver Formation has not been determined, but it is probably the same as the regional dip, about one degree or less to the southeast.

The overburden in the LSB area consists of Recent fill and Quaternary eluvial and alluvial deposits. The thickness ranges between 13.5 and 27.5 feet. Recent fill is present almost throughout the entire area and consists mostly of sludge removed from the LSB. The fill thickness ranges from 3 to 10 feet. The eluvial and alluvial materials consist mostly of poorly graded, silty, fine-grained sand with moderate amounts of sandy, silty clay and minor amounts of clayey sand, sandy clay, silty clay, and lean clay.

The contaminated aquifer is within the overburden and the material is essentially the same as that described above. The majority of groundwater movement occurs in unconsolidated, fine-grained sand and/or silty, fine-grained sand and clayey, fine-grained sand. The thickness of the aquifer ranges from 9.5 to 21.0 feet. The aquifer is unconfined and overlies the top of bedrock.

Contamination. Soil contamination in the LSB consists of raw materials, such as mustard agent production-related compounds; manufacturing by-products, such as volatile aromatic solvents; and degradation products from the synthesis of pesticides. Organochlorine pesticides that have been detected are dieldrin, aldrin, endrin and isodrin. Other contaminants detected were organosulphur compounds of chlorophenylmethyl sulfide, chlorophenylmethyl sulfoxide, and chlorophenylmethyl sulfone. DDT was also detected in an isolated area. Volatile organic compounds consist of chloroform, benzene, and chlorobenzene. The most prevalent metals are arsenic and mercury. Elevated concentrations of copper, lead, zinc, cadmium, and chromium were also detected.

Groundwater contaminants in the unconfined aquifer include volatile organic compounds, aromatics, metals, and organochlorine pesticides.

Arsenic, mercury, chromium, and copper are metals that have been detected in the ground water.

Decision Document Summary. The Interim Response Action for the LSB consists of moving the lime sludges currently located around the basins into the basins, a 360-degree subsurface groundwater barrier (slurry trench) around the basins to prevent migration of contaminated groundwater, a groundwater extraction system inside the isolation cell to maintain an inward hydraulic gradient, and a soil and vegetative cover over the cell to reduce infiltration of rainwater (Woodward-Clyde Consultants, 1990).

Pre-Design Field Investigations. Field investigations were conducted during June and July 1990. Investigations consisted of: electro-magnetic surveys for locating buried metallic objects (none were found); exploratory drilling and soil sampling in the LSB area; slug tests for hydraulic conductivity analysis; groundwater and tap water sampling; and bulk soil sampling of borrow areas. All investigations except the borrow investigations were conducted in level B personal protective equipment.

A total of 30 borings were drilled for this investigation. Nineteen borings were drilled along the alignment of the proposed slurry cutoff trench to identify the subsurface materials and to determine the consistency, density, and moisture content of the overburden; and also to determine the depth and characteristics of the claystone bedrock for design of the base of the proposed slurry trench. Eight borings were drilled outside the slurry trench area to further define the extent of the lime sludge material. Three wells were installed inside the slurry trench area for slug tests to determine the hydraulic conductivity of the overburden aquifer. Split-spoon samples were taken from all borings for geotechnical analyses, compatibility testing, and chemical analyses. All drill holes were backfilled with cement grout after completion.

Development of Laboratory Testing Methodology. In developing the MRDL's test equipment and procedures, various references were researched including work done by David J. D'Appolonia (1980), the U.S. Army Corps of Engineers Waterways Experiment Station (WES) (Zappi et al., 1989a, 1989b), the USEPA (JRB Associates, 1984), Dr. David Daniel (Daniel et al., 1984), and Goldberg-Zoino & Associates (GZA) (Ayres et al., 1983). The MRDL procedures were patterned after the work done in 1981 by GZA during design and construction of the Gilson Road Superfund Site cutoff wall. Procedural and equipment modifications were made at the MRDL based on early trial runs to address site specific conditions and speed up the overall test process. However, the basic concept of optimizing the mix design prior to long term compatibility testing was adhered to.

In reviewing the literature, there appeared to be no consensus on which type of permeameter, fixed wall or flexible wall, produced more realistic results. Each type of permeameter has its advantages and disadvantages and both can yield grossly misleading results under certain circumstances. Based on ease of operation and relatively expedient and reproducable results, fixed wall permeameters were selected for the mix design optimization phase. The flexible wall permeameter was selected for the long term compatibility phase because of its ability to accurately model various anticipated field stress conditions.

The equipment was designed and built at the MRDL with input from USACE engineers, technicians, and shop personnel. To prevent degradation of test equipment, anodized aluminum base and top caps, brass stones, stainless steel valves, teflon tubing, and glass burrettes were used. This allowed for multiple use of most of the equipment components after decontamination of the system prior to testing.

Backfill Soil Selection

To obtain a low permeability (typically 1 x 10-7 cm/sec or less is specified for completed soil-bentonite slurry trenches), soil with an appreciable amount of fines is necessary for the soil-bentonite backfill.

The USEPA recommends the following gradation criteria for backfill soils: maximum particle size of 5 inches, 65-100 percent passing 3/8 inch sieve, 35-85 percent passing the U.S. standard sieve #20, and 20-50 percent passing the U.S. standard #200 sieve. Plastic fines are preferred but not necessary (JRB Associates, 1984).

Soils excavated from the trench may be utilized for the backfill soil. This practice saves the time and money of locating, purchasing, developing, and hauling borrow soil to the site as well as disposal of the excavated soil. However, if the in situ soil is not suitable (for example coarse gravel) or is contaminated (as is often the case at HTW sites) imported borrow soil may be the only viable option.

Due to contamination of the in situ soil, the work plan called for testing of both in situ soil and a borrow source. Originally, a clay borrow area used in previous remediation projects at RMA was suggested. However, the clay borrow area is located in a bald eagle habitat which is closed to traffic from November 1 to April 1 and the amount of clay soil remaining is limited. Therefore stockpiles of soil excavated from the Lower Derby Dam spillway construction at the Arsenal were selected as the primary borrow soil. Soil from the clay borrow area would be used as a source of fines only, if necessary, to blend with either in situ or random fill borrow soil to achieve a low permeability.

Soil samples from several of the borings along the trench centerline were to be blended to form one composite in situ sample for mix design optimization and compatibility testing. During blending, however, the reddish brown soil developed a yellow staining over approximately 30 percent of the surface over one night. At that point Corps personnel decided not to consider the in situ soil for use in the trench or further testing because of potential field handling problems.

Figure 1 shows the grain size distribution and Atterberg limits for the random fill and clay borrow soils. The random fill soil contains more fines than EPA recommends. This is not considered to be a problem since a finer soil will make a low permeability easier to obtain.

Bentonite Selection

General. To obtain a general idea of the effect of site contaminants on bentonite, samples of the following four bentonites were obtained for this study:

S-5 Natural, Black Hills Bentonite, Rapid City, SD
BH-Natural, H&H Bentonite, Grand Junction, CO
Bara-kade 90 SP, NL Baroid, Houston, TX
Bara-kade 90, NL Baroid, Houston, TX (treated)

The Corps of Engineers' slurry trench guide specification requires use of premium-grade, ultrafine, natural sodium cation-based montmorillonite powders (Wyoming-type bentonite) that conforms to American Petroleum Institute (API) Specification 13A, Sections 5, 12 and 13 (API, 1990).

However, most commercially available bentonite is treated and conforms to Section 4, not 5 of API Specification 13A. Bara-kade 90 is the only bentonite studied which is treated and therefore conforms to Section 4 of API Specification 13A. Bara-kade 90 is the same bentonite as Bara-kade 90 SP, but one-quarter pound of a polymer is added per ton of bentonite to produce Bara-kade 90 (Anderson, 1991).

Free Swell Tests (McCandless and Bodocsi, 1987). "Free swell" is the increase in volume of a soil from a loose dry powder form when it is poured into water, expressed as a percentage of the original (dry) volume. Two grams (2.2 cubic centimeters) of bentonite is slowly poured into 100 milliliters (ml) of water, and the volume of settled solids is recorded after 2 and 24 hours. For this study, two tests were performed for each bentonite; one using tap water from the Arsenal and one using contaminated groundwater from the site. Table 1 shows results of the free swell tests. Percent

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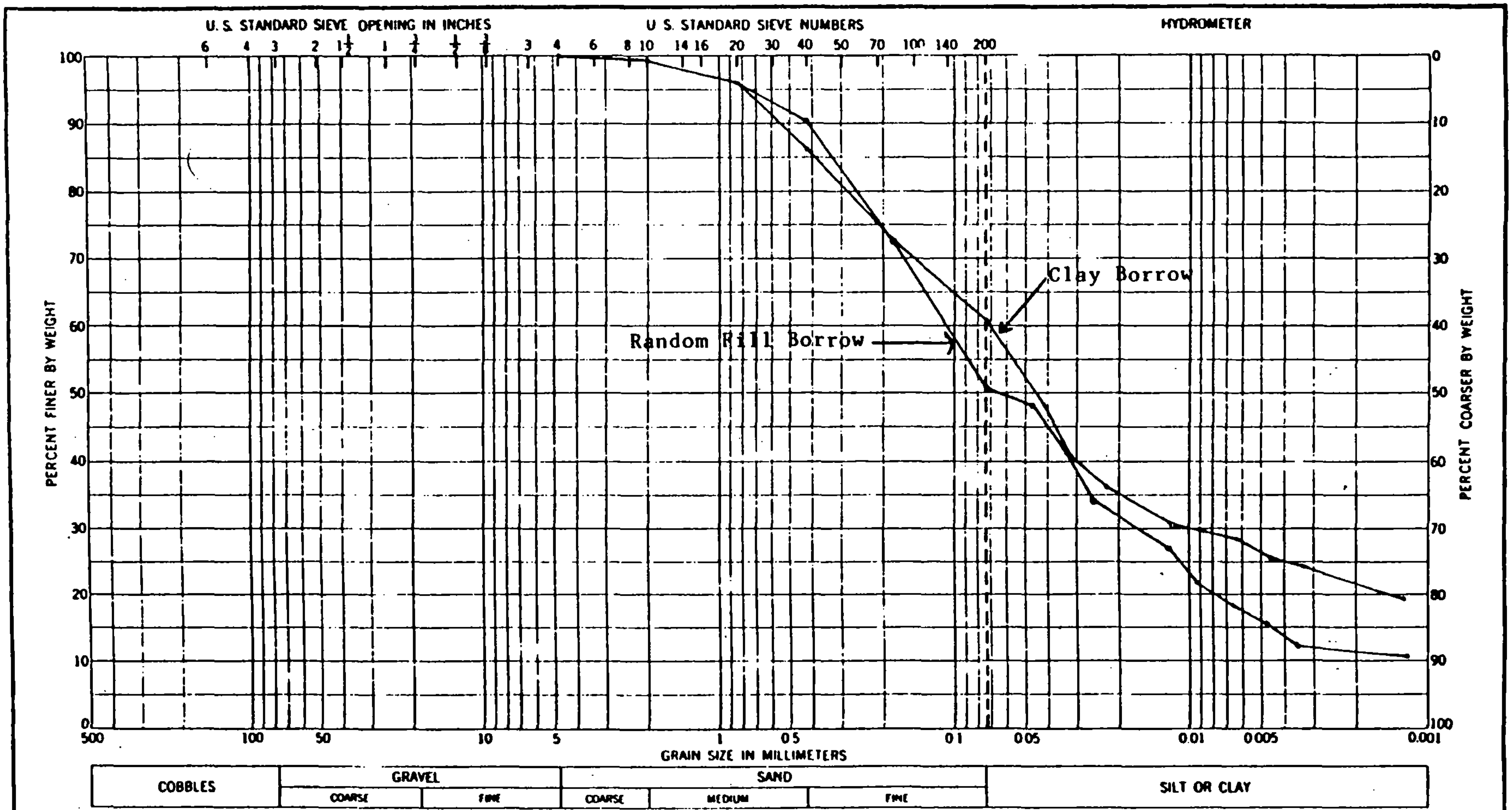

Sample No	Elev or Depth	Classification	Nat w %	LL	PL	PI
Random Fill Borrow		CL Sandy clay	17.6	42	22	20
Clay Borrow		CL Sandy Clay	35.0	35	13	22

Project: Lime Settling Basins Slurry Wall

Area: Rocky Mountain Arsenal

Boring No. Borrow Areas

Date: 30 November 1990

Figure 1. GRADATION CURVES

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24-hour swell is the percentage of the "final" (24 hour) swell achieved after 2 hours (tap water samples). Percent tap water swell is the percentage, at the given time, of the tap water sample swell achieved by the groundwater sample. Contaminants decreased the percent swell of all the bentonites, with Bara-kade 90 exhibiting the greatest decrease (about 50 percent). S-5 takes longer than the others to achieve "final" swell with both tap water and groundwater. The free swell behavior of BH-Natural and Bara-kade 90 SP is very similar, with Bara-kade 90 SP showing a slightly higher percent 24-hour swell after 2 hours and percent tap water swell with groundwater.

Filter Cake Compatibility Tests (D'Appolonia, 1980). As stated previously, the filter cake is an important component of a completed slurry trench. Filter cake permeabilities may be as low as 10-9 cm/sec (Xanthakos, 1979). For this reason filter cake compatibility tests, in addition to free swell tests, were used to evaluate bentonite performance. Slurry from each bentonite (prepared using RMA tap water) was placed in fixed wall permeameters. Slurry was forced through filter paper overlying a porous stone at the bottom of the chamber by a chamber pressure of 10 pounds per square inch (psi) for 24 hours. During this time a filter cake of approximately one-half inch formed on the filter paper. The bentonite slurry was removed with a vacuum bulb and immediately replaced with either RMA tap water or contaminated groundwater (one of each for each bentonite, for a total of eight tests). Water was forced through the filter cakes by a chamber pressure of 2-3 psi. The volume of effluent was measured two or three times a day for two to five days and the permeability was calculated.

The USEPA recommends the following properties for bentonite slurries: viscosity (measured with a Marsh funnel) greater than 40 seconds, unit weight around 65 pcf, pH between 7 and 10, and a bentonite content of 4 to 8 percent (JRB Associates, 1984). Millet and Perez (1981) recommend; viscosity greater than 40 seconds, unit weight around 65 pcf, and pH between 6.5 and 10. D'Appolonia (1980) recommends; viscosity greater than 40 seconds, and bentonite content of 5 to 7 percent. In this filter cake study all bentonite slurries were prepared with 6 percent bentonite by weight.

Marsh funnel viscosity, unit weight, and pH were measured for each slurry and are listed in Table 2. Properties of all slurries lie within the recommended ranges.

Figures 2 and 3 show results of filter cake compatibility tests. Some filter cakes formed cracks upon initiation of the flow phase of testing. After test completion, cutting the filter cakes into quarters revealed the cracks extended most or all the way through the filter cakes. However, presence of cracks did not appear to affect the permeability of the filter cakes. All bentonites except Bara-kade 90 SP exhibit a slight downward trend in permeability over time. Bara-kade 90 shows the least variation in permeability between tap water and groundwater. The reason for the drop in permeability of Black Hills S-5 (tap water) between 1390 and 1770 minutes is not known.

Selection. The original work plan called for selecting the bentonite which showed the least variation in filter cake permeability and percent swell between tap water and groundwater for use during further testing.

However, the bentonite which exhibited the least variation in filter cake permeability (Bara-kade 90) exhibited the most variation in percent swell. The designers eliminated Black Hills S-5 due to the drop in filter cake permeability in tap water between 1390 and 1770 minutes and Bara-kade 90 due to the large difference in percent swell between tap water and groundwater. Bara-kade 90SP was chosen because it shows slightly less variation in both percent swell and filter cake permeability between tap water and groundwater than BH-Natural and it shows a slight increasing trend in filter cake permeability over time. A 6 percent Bara-kade 90SP bentonite (by weight) slurry was used in all subsequent testing.

Table 1.

Free Swell Test Results

Bentonite	Time	Tap Water % Swell	% 24-Hour Swell	Ground Water % Swell	% Tap Water Swell
Black Hills	2 hr.	530	73	445	83
S-5	24 hr.	720		490	68
H&H Bentonite	2 hr.	785	91	560	71
BH-Natural	24 hr.	855		560	65
NL Baroid	2 hr.	785	83	400	51
Bara-Kade 90	24 hr.	945		400	42
NL Baroid	2 hr.	765	94	560	73
Bara-Kade 90SP	24 hr.	810		560	69

Table 2.

Bentonite Slurry Properties
Filter Cake Compatibility Tests

Bentonite	Marsh Funnel Viscosity (seconds)	Density (pcf)	pH
Black Hills	1. 48	64.9	8.7
S-5	2. 48		
	3. 48		
H&H Bentonite	1. 52	65.0	8.8
BH-Natural	2. 51		
	3. 52		
NL Baroid	1. 61	65.1	9.5
Bara-Kade 90	2. 62		
	3. 64		
	4. 64		
NL Baroid	1. 44	65.1	9.1
Bara-Kade 90SP	2. 44		
	3. 44		

Figure 2
Filter Cake Compatibility Test Results

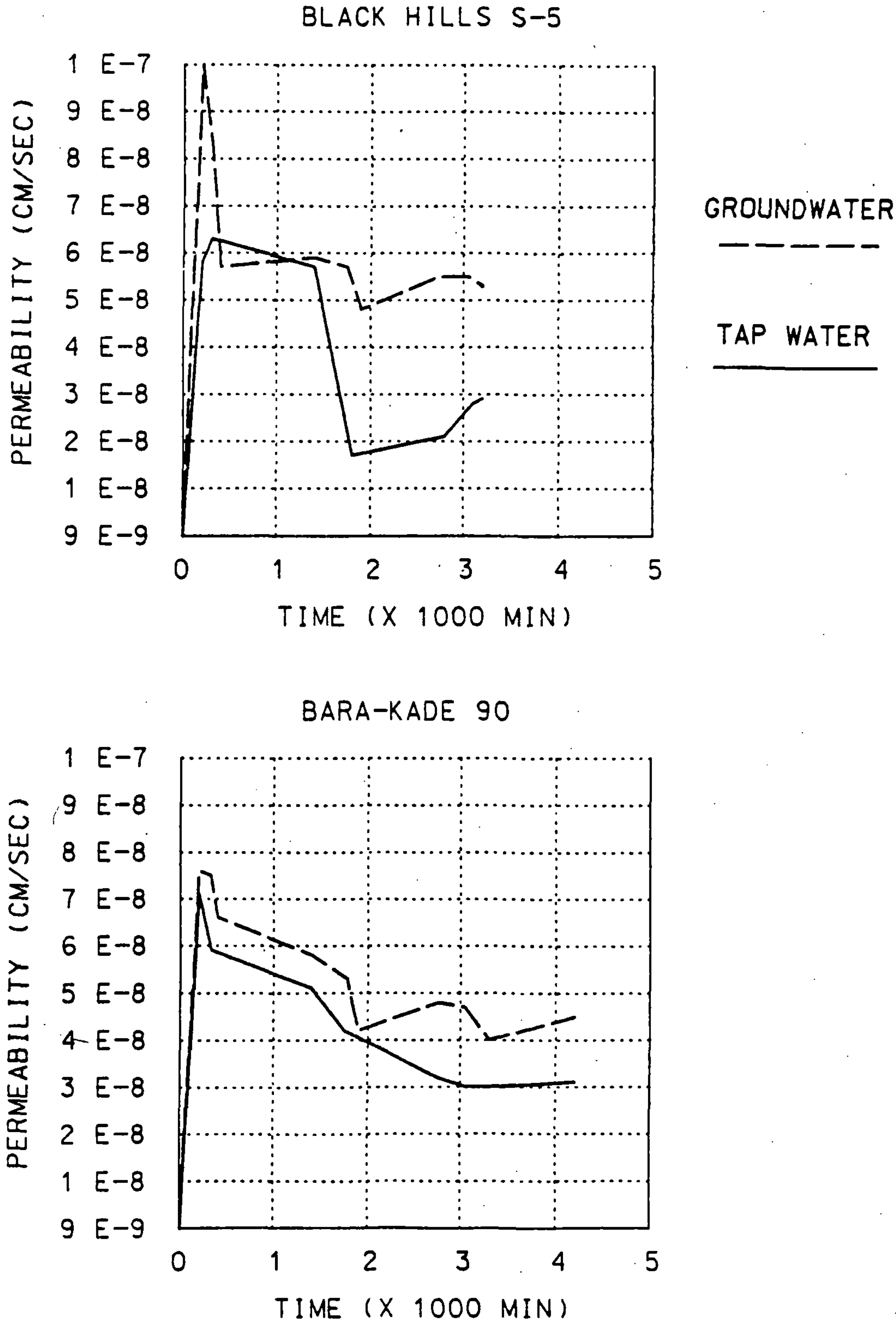
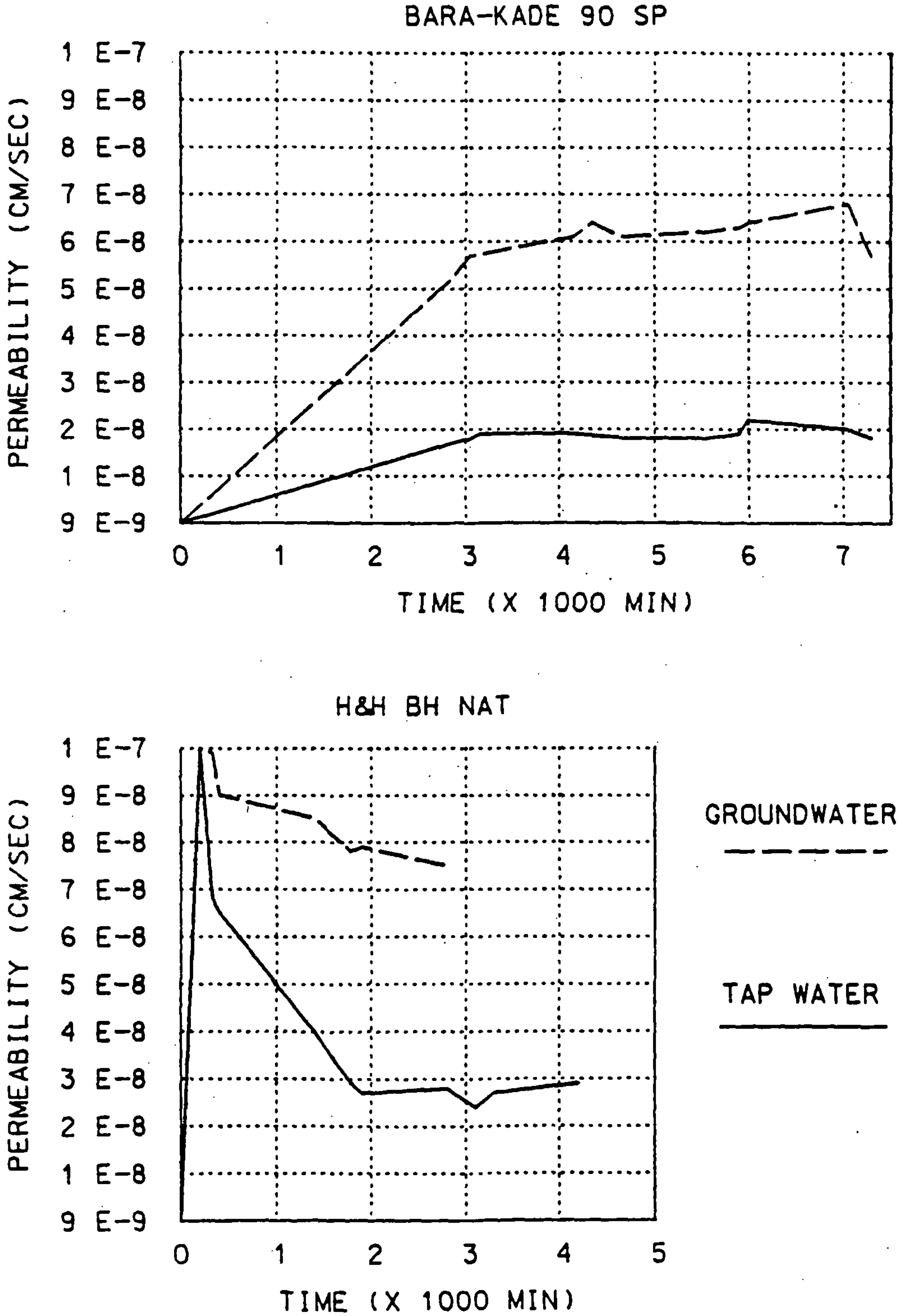


Figure 3

Filter Cake Compatibility Test Results



Mix Design Optimization

General. The purpose of this phase of testing is to determine the most economical mix of soil, dry bentonite, and bentonite slurry which will produce an in-place slurry trench permeability less than or equal to 1x 10-7 cm/sec. Because mixing and placing operations are less controlled in the field than in the laboratory, the designers specified a maximum laboratory permeability of 5 x 10-8 cm/sec for evaluation purposes.

Since borrow soil is available nearby at RMA, bentonite is the highest cost item. The HTW testing technical advisor assumed at some point it would be less expensive to decrease the permeability of soil-bentonite backfill material by adding additional fines (from a clay borrow area), rather than additional bentonite, to the random fill borrow soil. The "upper limit" bentonite content was set as 4 percent dry bentonite. Bentonite slurry is then added to the mixture to achieve a (concrete) slump between 4 and 6 inches.

Procedure. The work plan called for preparation of three samples of backfill soil with the addition of 0, 2, and 4 percent dry bentonite by weight. Bentonite slurry with a Marsh funnel viscosity of about 40 seconds is added to each sample to achieve a (concrete) slump of 4 to 6 inches. If fixed wall permeameter tests of 48 to 72 hours duration did not measure a permeability less than or equal to 5 x 10-8, clay borrow soil would be added to the random fill borrow soil to produce samples with approximately 10 percent greater fines content than the random fill borrow soil. The procedure (addition of dry bentonite and bentonite slurry, fixed wall permeameter tests) would be repeated. If measured permeabilities were still greater than 5 x 10-8 cm/sec, additional clay borrow soil would be added to produce samples with approximately 20 percent greater fines content than the random fill borrow soil. If measured permeabilities (after addition of dry bentonite and bentonite slurry) were still greater than 5 x 10-8 cm/sec, additional clay borrow soil would be added to produce samples with approximately 30 percent greater fines content than the random fill borrow soil.

Testing. The HTW testing technical advisor intended carrying out these tests in duplicate, using RMA tap water as the only permeant. The project designers misunderstood and requested one set of tests be performed using RMA tap water as permeant and one set be performed with contaminated groundwater as the permeant. In the first tests performed a few of the permeameters emptied of permeant over one night. The head pressures were 2 psi and the initial permeant volumes were approximately 200 ml. Examination revealed these specimens appeared to have contracted (specimens pulled approximately one-eighth of an inch away from the permeameter), pointing to a physical change as a result of some reaction with the permeant. To prevent preferrential flow of permeant between the permeameter wall and the sample, the permeameters had been coated with a bentonite paste (approximately 17 percent bentonite and 83 percent water by weight). The bentonite paste wall coatings were not evident at this point. These conditions occurred more frequently in the specimens permeated with contaminated groundwater, but also appeared in tap water permeated specimens as well. It was initially suggested that these failures may have been due to some lattice collapse in the bentonite resulting from ion exchange. The same or a similar process might possibly cause the cracks observed during filter cake compatibility tests.

The HTW testing technical advisor suggested attempting to discover the cause of the rapid permeant loss. In the interest of proceeding with testing, the advisor suggested, and the designers concurred, a triaxial permeability test be conducted using a 2 percent dry bentonite mix. Since the random fill borrow soil contains 51 percent fines and little difference exists in the grain size distributions of the two borrow soils (Figure 1), the addition of fines from the clay borrow soil would likely have a negligible effect on the permeability of the mix. Early results from a successful fixed wall permeability test indicated a permeability of approximately 5 x 10-8 cm/sec for a 2 percent dry bentonite mix.

While the triaxial test was being started, an investigation of the failed fixed wall tests was undertaken. Two paste coated jars, one filled with tap water and the other with contaminated groundwater were prepared. Several days of exposure to the liquids resulted in the tap water having a more detrimental effect on the paste than the groundwater. This was in contrast to the greater frequency of failed groundwater permeated fixed wall tests. Next, one still intact fixed wall test specimen was allowed to flow until the entire volume of permeant passed through it. Several hours later it appeared identical to the failed test specimens; the sample appeared to contract and the bentonite paste coating was missing.

This (very limited) investigation suggested that due to high permeability, cracking of the specimen, leakage along the permeameter walls, or a combination of the factors, permeant was forced through and/or around the specimen. Continued pressure application with no permeant caused drying of both the specimen and the bentonite paste. (The paste has a high water content (500 percent)). Drying could cause specimen shrinkage and give the appearance of a physical change due to some chemical reaction.

The HTW testing technical advisor thought not enough time was allowed between specimen set up and the start of flow. Persons at WES familiar with this type of testing concurred. All future fixed wall soil-bentonite backfill permeability testing will be run after incrementing the applied head pressures slowly over the course of several days.

Triaxial Permeameter Test Results. Figure 4 shows the results of the triaxial permeameter optimization test. The average permeability, approximately 4 x 10-8 cm/sec, is lower than the specified maximum of 5 x 10-8 cm/sec. Therefore the optimum mix design is 2 percent dry bentonite by weight and bentonite slurry added to the random fill borrow soil.

D'Appolonia (1980) recommends the following properties for soil-bentonite backfill material: slump between 2 and 6 inches, unit weight at least 15 pcf greater than the slurry unit weight, water content between 25 and 35 percent, minimum bentonite content of 1 percent, and a minimum fines content of 20 percent. Millet and Perez (1981) recommend a slump of 4 to 6 inches and a bentonite content of 2 to 4 percent. The USEPA recommends a bentonite content of 1 to 2 percent, water content of 25 to 35 percent, fines content of 20 to 60 percent, slump of 2 to 7 inches, and a unit weight at least 15 pcf greater than the slurry unit weight (JRB Associates, 1984). Table 3 lists physical properties of the triaxial permeameter specimen. All properties lie within the recommended ranges except water content. The reason for the high water content and it's effect on long-term permeability (if any) is not known.

<u>Long Term Compatibility Tests</u>

Flexible Wall Permeameter Equipment. The basic components of MRDL's flexible wall permeameter setup are: 1) Six modified triaxial permeameter cells, each consisting of anodized aluminum top and bottom cell bases, a clear lucite cylinder, anodized aluminum top and bottom specimen caps and brass porous stones; 2) Separate inflow and outlow glass burettes for flow quantity measurements; 3) Three pressure regulators with associated pressure gauges to control and monitor cell pressure, inflow, and outflow pore pressures; and 4) A stainless steel control panel with appropriate stainless steel valves, teflon tubing and spill containment tray. The LSB testing program utilizes air as a pressure source. For some permeant liquids, an inert gas (such as nitrogen) should be the pressure source to minimize biodegredation within the liquid.

Procedure. The test procedure can be broken down into six steps. The first step consists of forming a cylindrical specimen approximately 2.8 inches in diameter by 2.0 inches high out of the selected soil bentonite mix from the mix design optimization phase. This is done by using the bottom specimen

Figure 4

Triaxial Optimization Test

Borrow Soil and 2% Dry Bentonite

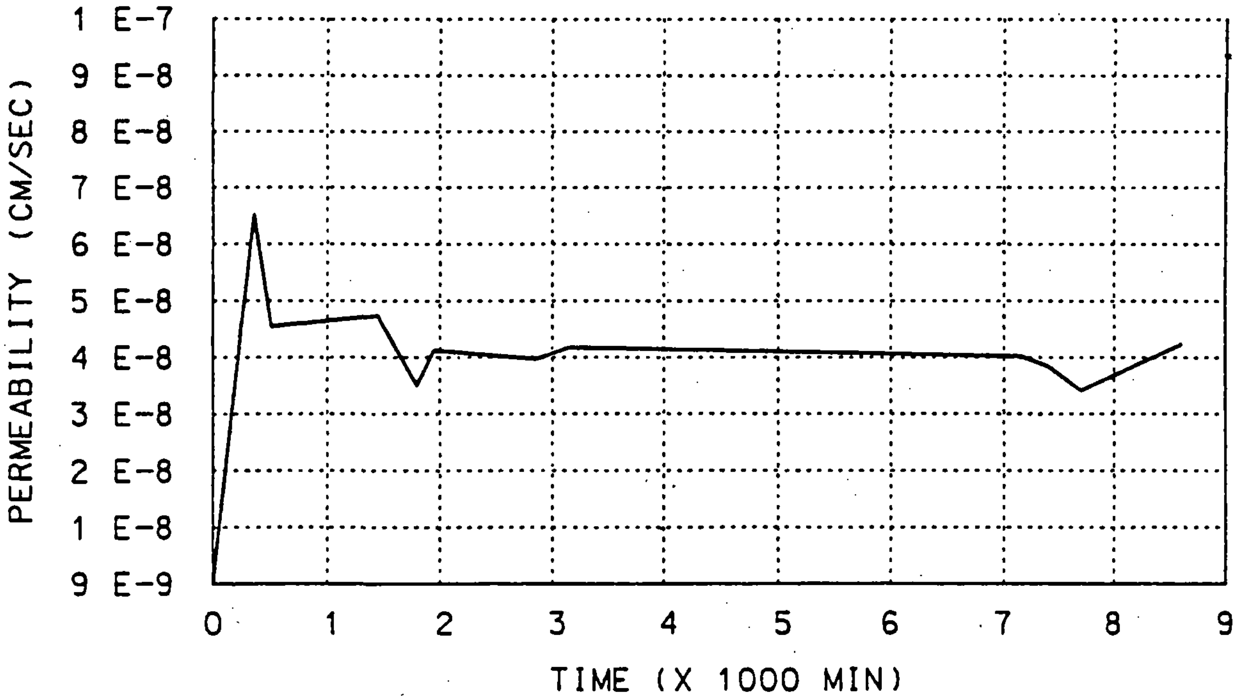


Table 3.

Physical Properties - Triaxial Optimization Test

Total Percent Bentonite	4.2 percent
Slump	6.125 inches
Wet Density	112 pcf
Dry Density	71.5 pcf
Saturation	100 percent
Void Ratio	1.35
Water Content	56.6 percent

Table 4.

Physical Properties - Compatibility Tests

	2% Dry Bentonite		4% Dry Bentonite
Property	Specimen 1	Specimen 2	Specimen 3
Total Percent Bentonite	3.7	3.7	6.0
Slump (inches)	4.5	4.5	5.75
Wet Density (pcf)	109	108	104.5
Dry Density (pcf)	73	72	67
Saturation (%)	100	100	100
Void Ratio	1.31	1.35	1.52
Water Content (%)	49.3	50.0	55.9

cap and a latex membrane sleeve within a perforated plastic cylinder as a specimen mold. Soil-bentonite backfill material is carefully spooned into the mold in two lifts and rodded lightly to produce a homogeneous low density mass. After taking the necessary specimen measurements and weights, top cap is set and the cell is assembled. Step 2 consists of filling the inflow and outflow burettes and porewater lines with site tap water and the chamber with deaired water after making the appropriate connections to the control panel. Step 3 consists of backpressure saturating the specimen. Step 4 consists of consolidating the specimen to simulate field stress conditions. Step 5 consists of flow initiation from bottom to top within the specimen using a relatively low hydraulic gradient (e.g. 28). Inflow and outflow quantities are monitored until the rate of inflow equals the rate of outflow for at least 5 consecutive daily readings. In addition, at least one pore volume of water must flow through the specimen prior to introducing site (contaminated) groundwater. As with tap water, groundwater inflow and outflow are monitored and the test is run until at least two pore volumes of groundwater pass through the specimen. The final step consists of removing the specimen, obtaining final weights, measurements, moisture contents etc. Three test conditions are being evaluated: two specimens of the "optimum" mix design of 2 percent dry bentonite and bentonite slurry added to the random fill borrow soil and one specimen with 4 percent dry bentonite and bentonite slurry added to the borrow soil. After one pore volume of tap water passes through the samples, two of them (one optimum mix sample and the 4 percent dry bentonite sample) will be leached with contaminated groundwater. Results of the two tests using groundwater as the permeant can be compared to see whether a backfill with a higher bentonite content reduces changes in backfill permeability over time. Occasional sampling and chemical analysis of the effluent permeant is done to determine the effectiveness of the soil-bentonite backfill material in preventing migration of contaminants through the specimen. It is recommended that the flow phase of the tests be run at least two months to provide meaningful results concerning the effects of the groundwater on the soil-bentonite backfill material.

Testing. Long-term compatibility testing began in early March 1991. Presently the first pore volume of RMA tap water is flowing through the specimens. MRDL personnel anticipate beginning groundwater permeation (for two of the samples) sometime during the week of April 1, 1991. Therefore, the effect of site contaminants on the permeability of the soil-bentonite backfill material is not known at this time. Tap water permeabilities are averaging between 4 x 10-8 cm/sec and 5 x 10-8 cm/sec, similar to values obtained during the mix design optimization phase. Table 4 lists physical properties of the test specimens. Water contents are higher than recommended values for (as yet) unknown reasons.

Chemical Tests

The small volume of effluent to be produced precludes performing a wide range of chemical testing. Sodium, calcium, and total organic carbon tests will be performed after each pore volume has moved through the samples. An increase in the amount of sodium and a decrease in the amount of calcium in the permeameter effluent could indicate displacement of sodium ions in bentonite by calcium ions from the groundwater.

CONCLUSIONS

The following list of conclusions is to be considered incomplete due to the ongoing compatibility tests.

General Testing Methodology

(1) When designing soil-bentonite slurry trenches through highly contaminated areas, at least one uncontaminated imported borrow soil should be investigated and tested for use in the soil-bentonite backfill material. If the in situ soil contains too many contaminants for use, mix design and compatibility testing of the borrow soil can continue without delay.

(2) Due to the variability of commercially available bentonites, several should be evaluated for suitablility with site tap water and contaminated groundwater. The evaluation process should include both free swell and filter cake compatibility tests.

(3) When soils used in soil-bentonite backfill material contain a significant amount of fines, addition of fines during optimization testing as planned in this study may not be necessary.

(4) During rigid wall permeameter testing the applied head pressure should be incremented slowly over several days.

LSB Backfill Mix Design

(1) Addition of 2 percent dry bentonite and enough bentonite slurry to achieve a concrete slump between 4 and 6 inches to the borrow soil produces a soil-bentonite backfill material with a laboratory permeability less than 5 x 10-8 cm/sec.

DISCLAIMER

This paper is not intended to address every conceivable HTW site condition or all possible applications of soil-bentonite backfill mix design and/or compatibility testing. Mentioned commercial products are not the only products of their kind available. Mention of trade names or commercial products does not constitute endorsement or recommendation for use.

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Jeffrey C. Evans[1]

HYDRAULIC CONDUCTIVITY OF VERTICAL CUTOFF WALLS

REFERENCE: Evans, J. C., "Hydraulic Conductivity of Vertical Cutoff Walls," Hydraulic Conductivity and Waste Contaminant Transport in Soil, ASTM STP 1142, David E. Daniel and Stephen J. Trautwein, Eds., ~~American Society~~ for Testing and Materials, Philadelphia, 1994.

Abstract: Vertical cutoff walls have been used to control the movement of contaminants and contaminated groundwater since the remediation of contaminated sites began. There are, however, significant hydraulic conductivity differences between soil-bentonite, cement-bentonite, plastic concrete, and in situ mixed cutoff walls. The results of laboratory and field studies were assessed to show the influence of material properties, confining stress, permeameter type, water table position, and state of stress, on the hydraulic conductivity of vertical cutoffs.

The results of these studies show the range of hydraulic conductivity expected for each of the cutoff wall types. Increasing confining stress markedly decreases the hydraulic conductivity of soil-bentonite and has a measurable but reduced impact on stronger backfill materials. Studies on soil-bentonite cutoff walls show that the stress at depth is less than predicted using the effective weight of the overlying materials. This reduction in stress is a result of soil-bentonite materials "hanging-up" on the side walls of the trench. Thus, applying the effective stress calculated from the effective weight of the overlying backfill overestimates the stress to be used in the laboratory tests and results in unconservative measures of hydraulic conductivity. Field data also reveals that, with time, the hydraulic conductivity of soil-bentonite above the water table may increase substantially. Further, the hydraulic conductivity does not significantly decrease upon re-saturation.

Keywords: slurry wall, hydraulic conductivity, soil-bentonite, cement-bentonite, plastic concrete

[1]Associate Professor of Civil Engineering, Bucknell University, Department of Civil Engineering, Lewisburg, PA 17837

INTRODUCTION

Vertical barriers are widely employed in the subsurface to control the flow of ground water and to reduce the rate of contaminant transport. Vertical cutoff walls have been used to control the movement of contaminants and contaminated groundwater since the remediation of contaminated sites began; one of first superfund sites where remedial technologies were implemented employed a slurry trench cutoff wall (Salvesen 1983). The principal factor in the performance of vertical barrier systems is the hydraulic conductivity. Like other geotechnical materials, there is no unique value of hydraulic conductivity. In cases where the ubiquitous value of 1×10^{-7} cm/s is specified, it is necessary to identify additional parameters which control this value in the laboratory and in the field. These parameters include the material composition, effective stress, field environment and cutoff wall defects. This paper will address the hydraulic conductivity of vertical cutoff walls with particular emphasis on soil-bentonite cutoff walls but including cement-bentonite, and plastic concrete slurry walls as well as in situ mixed walls (also known as deep soil mixed, auger mixed, soil mixed walls).

Soil-Bentonite Slurry Trench Cutoff Walls

The construction methods of soil-bentonite slurry trench cutoff walls are well-established (Spooner et al., 1984, Ryan 1987, Evans, 1993). A narrow (typically 0.5 to 1 m), slurry filled trench is first excavated in the subsurface. The slurry, comprised of a mixture of about 5% bentonite and 95% water by weight, is employed to maintain trench stability as the excavation proceeds downward from the ground surface. As the excavation proceeds longitudinally, the trench is backfilled by displacing the slurry with a mixture of soil, bentonite-water slurry, and occasionally dry bentonite. The soil used in the backfill may be soil excavated from the trench, borrow soil imported from offsite, or a mixture of both, depending upon grain size characteristics, the presence/absence of contamination and project hydraulic conductivity requirements. The hydraulic conductivity of soil-bentonite is typically between 1×10^{-7} cm/s and 1×10^{-8} cm/s. The excavation, backfill mixing, and backfill placement are shown schematically on Fig. 1.

Cement-Bentonite Slurry Trench Cutoff Walls

The construction methods of cement-bentonite slurry trench cutoff walls are also well-established (Spooner et al., 1984, Ryan 1987, Evans, 1993). A narrow (typically 0.5 to 1 m), slurry filled trench is excavated in the subsurface as with the soil-bentonite technique. The slurry in this case is comprised of a mixture of about 5% bentonite, 10% to 20% cement, and 75% to 85% water by weight. Cement-bentonite mixes have also incorporated fly ash as cement replacement (Carr 1990). In Europe, slag is commonly incorporated in the mix (Jefferis, 1981b). The slurry is employed to maintain trench stability and is left to harden in place to form the completed cutoff wall. Cement-bentonite may be the

cutoff wall of choice where strength considerations indicate the need for a material stronger than soil-bentonite. The hydraulic conductivity of cement-bentonite is typically between 1×10^{-5} cm/s and 1×10^{-6} cm/s and occasionally lower.

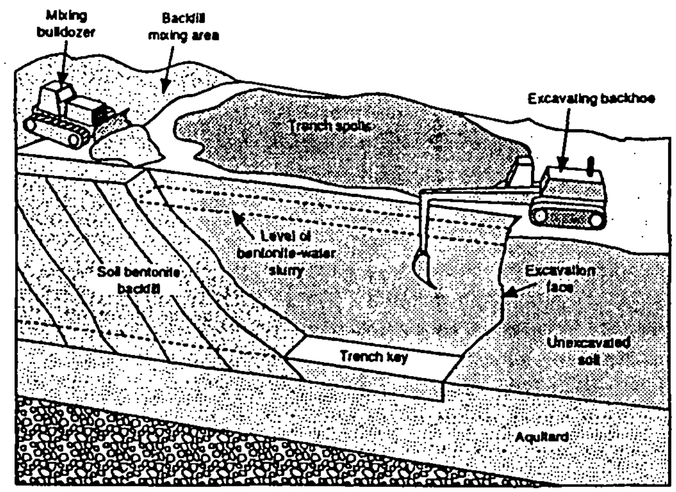

Fig. 1 Construction of a Soil-Bentonite Slurry Trench Cutoff Wall (from LaGrega et al. 1994)

Plastic Concrete Slurry Trench Cutoff Walls

Plastic concrete is a mixture of cement, aggregate, bentonite and water resulting in a material that is relatively strong with a relatively low hydraulic conductivity (Evans et al. 1987) Plastic concrete cutoff walls are usually constructed using the panel method of slurry trench construction. In this method of construction, the trench is excavated in panels using bentonite water slurry to maintain trench stability. The excavated panel is backfilled using plastic concrete placed using a tremie method of concrete placement. This panel excavation and backfill technique is similar that used for diaphragm walls (Xanthakos 1974). Although plastic concrete has been used in several applications, the higher cost when compared to soil-bentonite cutoff walls has limited its use. The hydraulic conductivity of plastic concrete barrier walls is typically between 1×10^{-7} cm/s and 1×10^{-8} cm/s.

In Situ Mixed Vertical Barriers

Using specially designed and fabricated augers, vertical barriers can be mixed in place. In-situ mixing is often called deep soil mixing (DSM) or a soil-mixed wall (SMW) process. Regardless of the name the process is similar; a special auger mixing shaft is rotated into the ground while simultaneously adding bentonite-water slurry or cement-bentonite-water slurry. The construction sequence shown in Fig. 2 results in a column of blended soil when multiple mixing shafts are employed. If additional strength is needed reinforcing can be added to the treated soil columns. The resulting wall is typically from 0.5 to 0.8 m wide. The bentonite-water slurry normally contains about 5% bentonite and 95% water. Mixing this slurry with the soil typically results in a wall with a bentonite content of about 1%. Since the wall is constructed as a mixture of the in situ soils, variability in the soil properties both along the wall alignment and with depth results in variability in the hydraulic conductivity of the completed wall. The hydraulic conductivity of in situ soil mixed walls is typically between $1x10^{-6}$ cm/s and $1x10^{-7}$ cm/s.

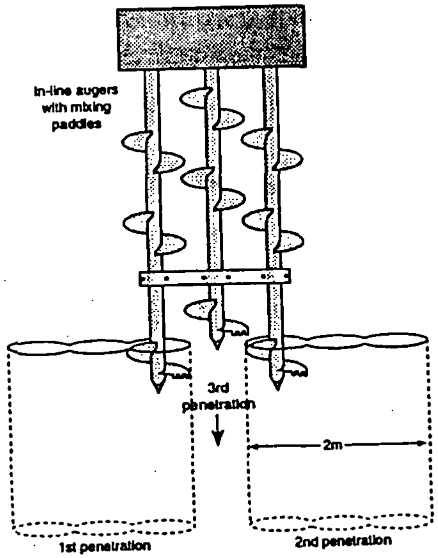

Fig. 2 Construction of an In Situ Mixed Cutoff Wall

PARAMETERS AFFECTING HYDRAULIC CONDUCTIVITY OF CUTOFF WALLS

What is the "true" hydraulic conductivity of a completed vertical barrier wall? How does the hydraulic conductivity of the wall relate to the hydraulic conductivity measured in the laboratory or in the field on samples of the wall? What are the factors which influence the hydraulic conductivity of the completed wall? What are the factors which influence the measurement of the hydraulic conductivity of the cutoff wall material? Without attempting to revisit all the factors involved in hydraulic conductivity testing, the remainder of this paper will focus on several parameters which influence the hydraulic conductivity of the vertical barrier walls described above.

Parameters which influence the hydraulic conductivity and our measures of hydraulic conductivity include:

1) grain size distribution
2) bentonite content, type and gradation

3) effective consolidation pressure in the laboratory
4) field state of stress
4) homogeneity of the cutoff wall
5) hydraulic fracturing
6) permeameter type
7) location of the water table
8) variability
9) nature of the pore fluid and permeant
10) potential for defects

Effect of Grain Size Distribution

It has long been established that the type and nature of the fines fraction influences the hydraulic conductivity of the soil-bentonite backfill (D'Appolonia 1980) In general, as the fraction of the soil finer than the No. 200 sieve increases, the hydraulic conductivity decreases. Shown on Fig. 3 is the relationship between the hydraulic conductivity and the fines content for the soils of a specific project. The data demonstrate the importance of "adequate" natural fines in achieving a low hydraulic conductivity. The low hydraulic conductivity is achieved without enriching the mix with additional dry bentonite. For this particular study, the addition of 20% plastic fines from a clayey borrow source to the base soil of about 20% gravel, 70% sand, and 10% silt, lowered the hydraulic conductivity to $5x10^{-8}$ cm/s, below the project target of $1x10^{-7}$ cm/s. The mixture using virtually 100% plastic fines from the borrow source resulted in a hydraulic conductivity of $3x10^{-8}$ cm/s, not significantly lower that for the mix containing only 20% natural fines.

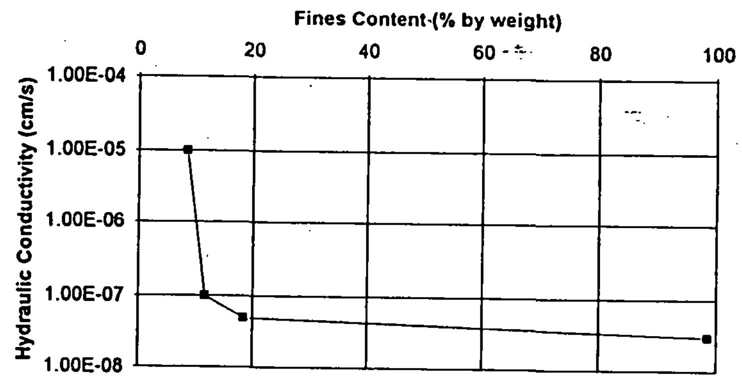

Fig. 3 Site Specific Relationship between Fines Content and Hydraulic Conductivity

The results shown in Fig. 3 were determined for specific soils from a specific site. Although there is clearly a relationship between fines content and hydraulic conductivity for these materials, that is not to say the relationship may be generalized. In fact, the data presented by Ryan (1987) in updating a relationship published earlier by D'Appolonia dismissed the notion that one can achieve a certain hydraulic conductivity by simply choosing a fines content.

These data are presented to illustrate the approach to determining the desired optimum soil-bentonite backfill mixture. A well-graded soil, consisting of a blend of gravel, sand, silt and clay results in a backfill of low hydraulic conductivity, low compressibility, and as discussed later in this paper, greater resistance to degradation by contaminants than a backfill containing a very high percentage of fines in the mixture. This approach is shown schematically on Fig. 4. The Figure shows the arrangement of progressively finer particles plugging progressively finer voids, leaving only the smallest voids to be filled with the clayey fines and the bentonite which is added via the slurry. The natural analogy to this approach is glacial till, generally well-graded and having a low hydraulic conductivity. Although segregation of the larger particles is theoretically possible, grain size distribution data indicates that the gravel remains well-distributed throughout the backfill.

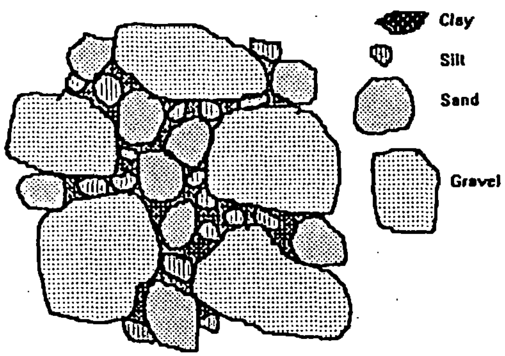

Fig. 4 Schematic of Well-Graded Soil

An added benefit results from a well-graded backfill when contamination resistance is taken into account. This will be discussed in more detail later in this paper.

Effect of Bentonite Content

Generally speaking, increasing the bentonite content in a vertical barrier will decrease the hydraulic conductivity in soil-bentonite and in situ soil mixed walls; there may , however, be an optimum. Shown on Fig. 5 is a relationship between hydraulic conductivity and the bentonite content. The data reveal that, for this particular mix, the minimum hydraulic conductivity was found at a bentonite content of about 3%. Although such correlations may be developed for site specific use, when data from about thirty soil-bentonite projects were combined, little correlation of hydraulic conductivity to bentonite was found (Ryan 1987). The same conclusion can be reached for cement-bentonite cutoff walls.

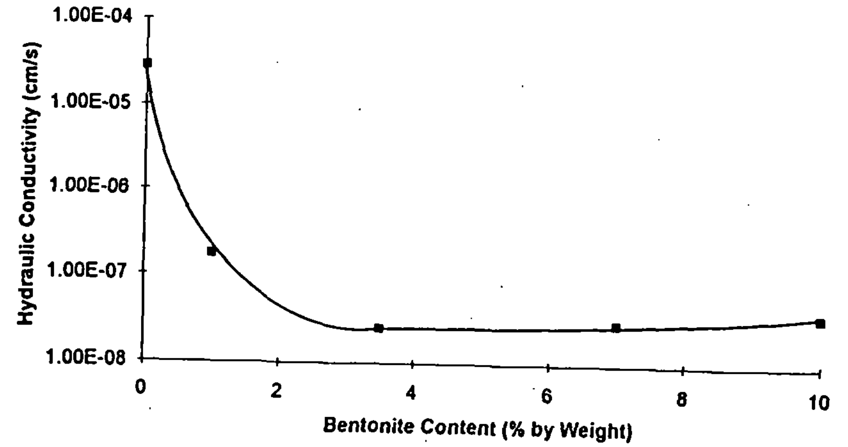

Fig. 5 Effect of Bentonite Content on Hydraulic Conductivity of a Soil Bentonite backfill (after Barvenik 1992)

Effect of Consolidation Pressure

For any give sample of vertical barrier material, the hydraulic conductivity decreases as the effective consolidation pressure increases. This trend is predictable on a theoretical basis from considerations of decreasing void ratio with increasing effective stress. Shown on Fig. 6 are relationships between effective consolidation pressure and hydraulic conductivity showing significant decreases in the hydraulic conductivity of soil-bentonite backfill as the effective consolidation pressure is increased.

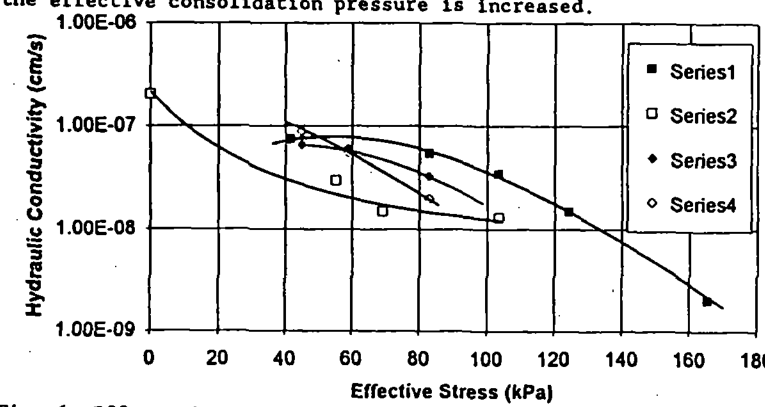

Fig. 6 Effect of Consolidation Pressure on Hydraulic Conductivity of a Soil-Bentonite Backfill (data from McCandless and Bodocsi 1988; Day 1992)

The impact of confining pressure in a laboratory permeability test may go beyond that expected from void ratio considerations. Shown in Fig. 7 are the results of a series of laboratory tests on molded samples of an in situ mixed wall of soil, bentonite and cement. The trend is evident, as the confining pressure increases, the hydraulic conductivity decreases. The authors conclude that, as a result of the rough surface of the cemented samples, a high confining pressure is needed to maintain contact between the membrane and the sample to prevent sidewall leakage.

Using the authors' data, the relationship between the confining pressure and void ratio is shown on Fig. 8. As shown, the decrease in void ratio due to the increasing confining pressure is quite small.

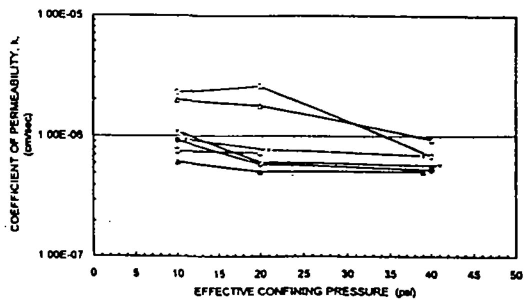

Fig. 7 Effect of Confining Pressure on the Hydraulic Conductivity of an In Situ Mixed Wall (after Yang 1993)

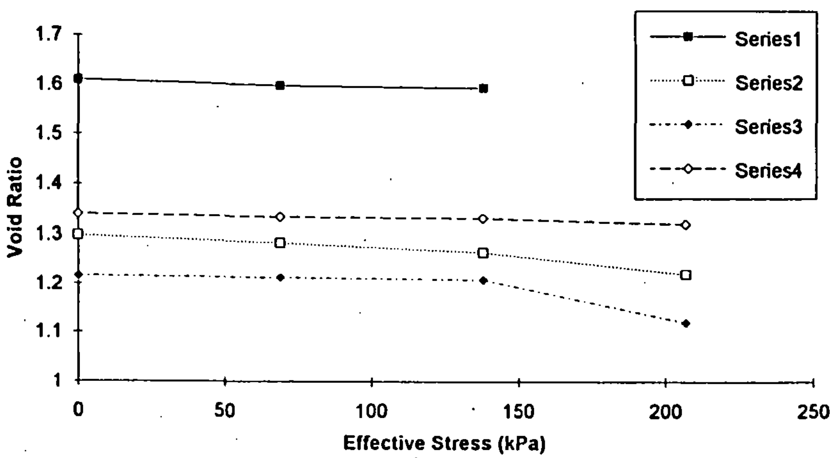

Fig. 8 Effect of Confining Pressure on the Void Ratio of an In Situ Mixed Wall (data from Yang, 1993)

Field Stress Conditions

The study of the influence of effective consolidation pressure on hydraulic conductivity is more than academic. Unless the state of stress in the field is known, the hydraulic conductivity remains uncertain. Laboratory model and field data obtained to date on soil-bentonite slurry trench cutoff walls indicated that the stress does not increase hydrostatically with depth (McCandless and Bodocsi 1987; Cooley 1991). Analytical approaches reveal similar findings (Sweidan, 1990). Data on a fully instrumented soil-bentonite cutoff wall measuring total and effective stress with depth was not found in the published literature. The soil-bentonite backfill is quite compressible compared with the relatively rigid trench sidewalls; as a result the consolidation of the backfill is limited by the friction at the trench/backfill interface, termed arching by some authors (Millet et al. 1992). Based upon the information available to date for soil-bentonite walls, the effective stress distribution with depth depends upon:

1) the wall thickness (or thickness/depth ratio),
2) the backfill compressibility,
3) the backfill/trench wall interface friction, and
4) the backfill density
5) poisson's ratio

The arching or reduction in effective stress at depth can be minimized by increasing the wall thickness or reducing the backfill compressibility.

Hydraulic Fracturing Effects on Hydraulic Conductivity

The nature and potential for hydraulic fracturing in soil-bentonite slurry trench cutoff walls is often misunderstood. Handbook guidance quotes a rule of thumb of 1 psi (of excess hydraulic pressure) per foot of wall depth as "safe against hydraulic fracturing" (USEPA 1984). That is, at a depth of 20 feet (6.1 m), the wall can withstand an excess hydraulic head of 20 psi (138 kPa). Alternatively, a width of 15 to 23 cm (0.5 to 0.75 ft.) per 3 m (10 ft.) of hydrostatic head is cited (Case 1980). For slurry wall use in waste containment applications, the guidance has been incorrectly interpreted to calculate the maximum drawdown from within the barrier. The guidance was originally developed for pore water pressure in excess of the original pore water pressure (i.e. pressure above hydrostatic) as in the case hydrofracturing rock to enhance oil recovery by pumping fluid into the formation at pressures large enough to reduce the effective stress to zero and "lift" the rock. This guidance is also applicable to the case of a slurry wall beneath the core of a dam where the upstream reservoir induces high hydraulic head. In such an application, the excess pore water pressure could exceed the minor principal total stress within the cutoff wall, reducing the minor principal effective stress to zero and resulting in hydraulic fracturing. Dewatering from within a cutoff wall lowers the phreatic surface relative to the original phreatic surface and results in an increase in effective stress within the wall. Since dewatering from within the cutoff wall cannot cause a reduction in minor principal effective stress, hydraulic fracturing can not result from dewatering.

Laboratory Permeameters and Their Influence on Hydraulic Conductivity

Much has been written regarding laboratory test methods and equipment and their effect on hydraulic conductivity values (Olson and Daniel 1981). The discussion here is limited to those unique equipment considerations that influence the hydraulic conductivity of vertical barriers. In particular, a fixed wall API Filter Press (API 1984) has been used to conduct rapid evaluations of hydraulic conductivity in the field as the construction progresses. Traditional fixed wall permeability tests have also been used. The data shown on Fig. 9 indicate a some correlation between the API filter press fixed wall test method and the triaxial test methods for two particular projects. These data show the need for site specific correlations if fixed wall permeability tests are to be used for field quality control tests.

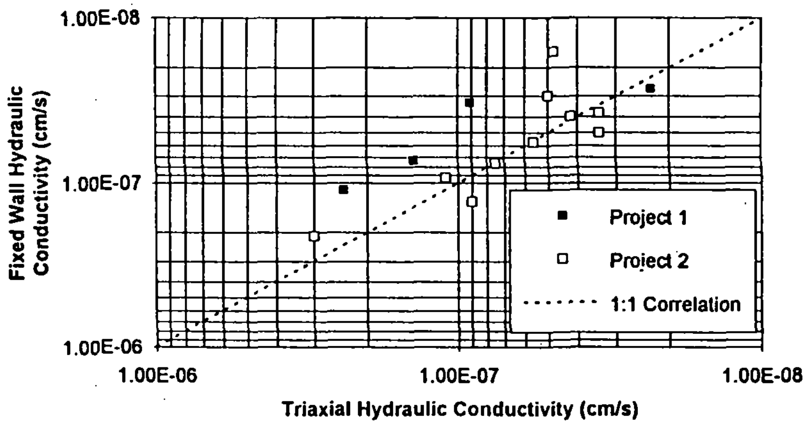

Fig. 9 Permeameter Test Results (data from Barvenik 1992 and Day 1992)

Influence of Fluctuating Water Table

The principal purpose of a cutoff wall is typically to impede the horizontal flow of ground water (and the associated transport of contaminants in many environmental applications). Considerable effort is made to hydrate the bentonite in the cutoff wall in an effort to minimize the hydraulic conductivity. Further, it is common for the wall to have a portion that is expected to remain permanently below the water table, another portion permanently above the water table and a portion which may be in the range of a fluctuating water table. Limited information on the long-term performance of cutoff walls is available, however, in one recent study measurements of permeability were made for a soil-bentonite cutoff wall that was constructed in 1981 and another constructed in 1987 (Cooley 1991). The cutoff walls joined to form a vertical cutoff surrounding a wet ash handling facility that maintained

essentially constant water levels year-round. The investigator obtained thin-walled tube samples above and below the water table for each of the different age cutoffs. The results are as shown in Table 1.

Table 1 - Effect of Water Table and Wall Age on Hydraulic Conductivity of Soil-Bentonite

Construction Date	Position w.r.t water table	Hydraulic Conductivity (cm/s)
1981	above	$1x10^{-3}$
1981	below	$1x10^{-8}$
1987	above	$6x10^{-6}$
1987	below	$1x10^{-7}$

To further investigate the potential for "rehydration" the permeability tests were repeated after 16 days of backpressure saturation with no change in the results. To examine the phenomena further red dye was introduced to see if the increased hydraulic conductivity could be attributed to defects in the sample. After permeation the samples were cut apart and examined; no dye paths were observed and the samples was noted to be uniform in cross-sectional appearance. Although these data are limited and the time spent rehydrating the clay was limited to 16 days, they give rise to concern that if soil-bentonite materials are not kept saturated, the hydraulic conductivity may increase and such increases are not reversible.

Variability of Cutoff Walls

As described above, the construction of these cutoff walls typically employs the in-place mixing of natural soils with bentonite, bentonite water slurry, and/or cement-bentonite slurry. It has also been established that the hydraulic conductivity is a function of the properties of the base soil to be blended (i.e., grain size-distribution, plasticity, water content, fines content). As a result, it is expected that the variability in the hydraulic conductivity of the completed soil-bentonite or in situ mixed barrier would be a function of the variability of the soils along the cutoff wall alignment. Thus, it is important to fully characterize the distribution of materials with depth and along the trench alignment in order to properly predict the range of hydraulic conductivity to be expected.

Perhaps expectedly, the test values of hydraulic conductivity of the completed cutoff wall depends on the method of sampling and testing. For one study of an in situ mixed soil, bentonite and cement barrier (Yang et al. 1993), the data ranged from a low of about $1x10^{-8}$ cm/s to a high of $1x10^{-4}$ cm/s. About 45% of the data meet the project requirements of $1x10^{-6}$ cm/s. A number of parameters were found to affect the laboratory test results. Thin wall samples were affected by damage to sampling tubes including cutting edges both during and after sampling. Soil-cement samples were observed to exhibit rough and loose surface zones and cracking. In essence, the scatter in permeability test data is attributed to the inferior quality of bulk samples and sample disturbance of core samples. In contrast to the laboratory data on both field and laboratory prepared and obtained samples, all of the data obtained from in situ permeability tests met the project requirements of 1×10^{-6} cm/s. The case study just described (Yang et al. 1993) suggests a need to develop more reliable methods for sampling in situ mixed soil/cement/bentonite materials and for determining the hydraulic conductivity testing of these materials as part of the construction quality control process.

Effect of Pore Fluid and Permeant

The hydraulic conductivity of soil-bentonite backfill can be altered as a result of permeation with permeants having a different chemistry than the original pore fluid. The practical questions are two; will the hydraulic conductivity increase or decrease and what will the magnitude of the change be? The nature of clay-pore fluid interactions has been well studied (Mitchell 1976; Evans et al. 1985; Brown and Anderson 1983). It is generally considered that the behavior of soils in the presence of contaminants can be modeled by the clay-water-electrolyte model as developed for colloidal suspensions (Mitchell and Madsen, 1987). In general, little effect is observed for clays permeated with chemicals at low concentrations. In contrast, permeation with concentrated organics may result in significant increases in hydraulic conductivity. Thus, to minimize detrimental clay/contaminant interactions it is important to minimize 1) the activity of the clay fraction, and 2) the amount of the clay fraction.

To meet these goals it is necessary to include only enough low plasticity clay to reduce the hydraulic conductivity to the desired level and to include only the quantity of bentonite which is mixed in by virtue of the addition of bentonite-water slurry for workability. Thus, for the schematic shown in Fig. 4, the gravel, sand, and silt components are virtually non-reactive and the slightly reactive low-plasticity clay is present in the minimum quantity necessary to achieve the desired hydraulic conductivity. In this way, the potential for major changes in the hydraulic conductivity due to incompatibility with the surrounding ground water environment are minimized.

Indicator tests such as sedimentation tests, cracking pattern tests, and/or Atterberg limits may be used to initially evaluate the potential for long term compatibility problems or short term construction problems (Alther et al. 1988). Compatibility testing should ultimately include a long-term triaxial permeability test using the expected leachates/permeants (Evans and Fang 1988). Although passing of two to three pore volumes of the contaminant is usually considered sufficient to investigate compatibility, recent research has shown that the permeant volume needed is dependent upon the contaminant mass needed to complete the reaction (Jefferis 1992).

Limited data indicate that plastic concrete may be less susceptible to changes in hydraulic conductivity when permeated with contaminated permeants than soil-bentonite (Evans et al. 1987).

Based upon the research to date, the presence of non-aqueous phase liquids may pose the greatest risk to the degradation of vertical cutoff walls. For additional detail regarding the compatibility of slurry cutoff wall materials the reader is referred to Day in this same proceedings Day 1993).

Potential for Defects in Vertical Cutoff Walls

No discussion of the hydraulic conductivity of vertical barriers would be complete without mention of the potential for defects, i.e. areas of high hydraulic conductivity. A defect is defined as that portion of the cutoff wall where the hydraulic conductivity is beyond the limits of that expected due to the statistical variability of the cutoff wall materials. The potential defects in slurry trench cutoff walls are many and have been described elsewhere (Evans 1993; Evans 1990; McCandless et al. 1993). The probability that any given defect will be detected in any given verification testing program is small. Most testing programs use laboratory tests of field prepared samples to verify the hydraulic conductivity of the cutoff. Even where field tests are used, it may not be economically feasible to conduct enough in situ permeability tests to reduce the probability of missing a defect to a reasonably small number. Non-destructive geophysical techniques have also been considered (Barvenik and Ayers 1987). Pumping tests may be used but in situ heterogeneity often precludes definitive conclusions regarding the integrity of the completed barrier. Recent studies show that the use of standpipes along the wall alignment may provide useful information if properly spaced (Bodocsi et al. 1993). Further research in this area is needed to better verify the as-constructed condition of vertical barrier walls.

THE FUTURE OF BARRIER TECHNOLOGIES

There is little doubt that advances will be and are being made along several fronts. These include construction techniques, design and analysis methods, laboratory and insitu testing methods, and in the philosophy of application. It is this last topic that perhaps offers the most promise. Historically, barriers have been constructed as the title of this paper reflects, as hydraulic barriers. However, it is understood and recognized that the ultimate goal may be more precisely stated as contaminant transport barriers. Thus there is a need to develop barrier techniques that are improved methods of reducing contaminant transport. This can be done by either further reducing the hydraulic conductivity or increasing the attenuation capacity of the barrier. Thus, HDPE membranes are being placed in cutoffs to achieve low hydraulic conductivity. The use of attenuating materials in the barrier system is also under study (Evans et al, 1990; Mott and Weber 1989, 1991).

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Steven R. Day[1]

THE COMPATIBILITY OF SLURRY CUTOFF WALL MATERIALS WITH CONTAMINATED GROUNDWATER

REFERENCE: Day, S. R., "The Compatibility of Slurry Cutoff Wall Materials with Contaminated Groundwater," Hydraulic Conductivity and Waste Contaminant Transport in Soils, ASTM STP 1142, David E. Daniel and Stephen J. Trautwein, Eds., American Society for Testing and Materials, Philadelphia, 1994.

ABSTRACT: Slurry cutoff walls are frequently relied upon to block groundwater flows from toxic waste sites and landfills. The long-term effectiveness of slurry cutoff wall materials is critical to the successful containment of these facilities and the protection of groundwater resources. A variety of laboratory indicator tests have been attempted by engineers and academia to make compatibility determinations but at present there has been little published experience to show which tests produce meaningful results and how these tests can be used to demonstrate compatibility.

Hydraulic conductivity is a useful measure of chemical/soil compatibility but permeability tests alone cannot assure the long-term stability of a slurry cutoff wall. A suite of indicator tests are used where the leachate and the proposed materials are combined and tested in immersion, desiccation, sedimentation, and other modes. Each indicator test attempts to model a different scenario of the slurry cutoff wall installation and operation.

This paper presents the experience of a specialty contractor from a number of projects, where an incompatibility was discovered and alternate materials were used to find a successful solution. Monitoring results from these sites has proven the effectiveness of the chosen solution. The laboratory test methods described are relatively simple and rely on worst-case scenarios, performed in a step-by-step process, that culminates with flexible wall permeability tests. Based on the methods described and the results from successful projects where these methods were used, engineers, owners and the public may better rely on long-term slurry cutoff wall performance with an increased level of confidence.

KEYWORDS: attapulgite, bentonite, compatibility, containment, deep soil mixing, hydraulic conductivity, slurry cutoff wall

[1]District Manager, Pittsburgh Office, Geo-Con, Inc., 4075 Monroeville Boulevard, Pittsburgh, PA 15146

STP 1142

INTRODUCTION

Slurry cutoff walls are permanent subsurface structures used to direct and control groundwater flow. Since the inception of this technique in the 1940's, slurry cutoff walls have been used where relatively unpolluted groundwater was diverted for civil works such as dams, dikes and dewatering structural excavations (Ressi di Cervia 1992). With the beginnings of CERCLA legislation and the environmental movement in the 1970's, more and more slurry cutoff walls are built to contain contaminated groundwater at landfills, hazardous waste and industrial facilities (Ryan 1987). The hydraulic conductivity or permeability of slurry cutoff walls is usually the performance criterion relied upon in the design, construction and contracting of these structures. For projects with an environmental function, the lowest practical hydraulic conductivity is typically specified for the maximum protection of the public and groundwater resources.

Hydraulic conductivity (permeability) testing has significantly improved over the last decade but is of limited use in determining incompatibility. The time and expense required for hydraulic conductivity tests limit the user in formulating compatible mixtures and complicates feasibility estimates. Furthermore, the flexible wall permeability test, the industry standard, requires the imposition of a confining stress, which can mask certain incompatibilities (Evans 1993).

In this paper, compatibility is defined as when two materials, i.e., contaminated groundwater (or leachate) and soil-bentonite, can be mixed together or coexist without reacting chemically or interfering with the performance of the soil-bentonite. An incompatible result is an increase in permeability in the soil-bentonite or chemical reaction which produces a degradation in the physical properties of the soil-bentonite.

Predetermining the compatibility of slurry wall materials with contaminated groundwater is generally recognized as good engineering practice (Ryan 1987; D'Appolonia 1980; Grube 1992; Millet and Perez 1981; Tallard 1984). Some methods, other than hydraulic conductivity testing, have been proposed to determine compatibility (McCandless and Bodocsi 1988; Khera and Thilliyar 1990; Wu and Khera 1990) but these have had limited experience and the results of some test are poorly understood. This paper presents a suite of relatively simple and quick indicator-type tests which can be used in concert with hydraulic conductivity tests to more quickly and better determine the most applicable materials for the containment of contaminated groundwater with slurry cutoff walls.

PURPOSE OF COMPATIBILITY TESTING

Compatibility tests should simulate the long-term, worst-case performance of slurry walls in a contaminated groundwater environment. As yet, no standards exist which can guide the user to determine compatibility.

The primary reason for performing compatibility tests is to ensure that the slurry cutoff wall performs as intended. Compatibility testing also makes the planning and construction effort more efficient and results in a higher quality installation. The most important reasons for completing compatibility tests are as follows:

1. ensure permanence of the materials,
2. estimate long-term performance,
3. estimate material and additive types and amounts,
4. ensure success of construction,
5. accelerate feasibility studies, and
6. address regulatory concerns.

In general, incompatibilities result from chemical reactions. It may be assumed that superior knowledge of the chemicals involved will preclude compatibility testing but practical experience has shown the current state of knowledge to be limited (Ryan 1987). In some cases (e.g. landfills) the types and concentrations of chemicals varies widely. On other sites with more definable chemistry, the subspecies which result from mixing with groundwater cause similar uncertainty. Therefore, while a thorough understanding of soil/waste chemistry is important, studies to detect incompatibilities must rely on experimental methods.

It is, therefore, the purpose of this paper to explain and illustrate, by example, tests which can be used to determine the gross compatibility or incompatibility of slurry cutoff wall materials when used in contaminated groundwaters.

POTENTIAL FAILURE MECHANISMS

Slurry cutoff walls are susceptible to failure during construction and operation as a result of groundwater contamination. Because of the specialized nature of the construction process, the materials selected for the installation must meet workability restraints. In practice, this means that the materials must be suitable for the specialty contractors' requirements as well as the designers objectives for the installation to be effective.

The first and most important ingredient in slurry cutoff wall construction is the bentonite slurry. Ineffective slurry results in excessive material usage, the necessity for additives and/or the loss of slurry workability. Fresh water for mixing and premium grade bentonite are the primary slurry ingredients. Poor quality water (e.g. hard or polluted water) and/or poor quality bentonite can usually be identified by testing trial mixtures.

Excavating through refuse or concentrated wastes can have a detrimental effect on slurry performance. Unusual or excess material usage can result. Flocculation of bentonite in a slurry trench will often result in a trench collapse and/or massive settlement of solids on the bottom of the trench which limits backfilling. Contaminated groundwater has been a cause of bentonite flocculation and, therefore, tests to predetermine the potential for construction failures, material usage estimates and the need for additives is critically important.

Contaminants may react with the key ingredient, bentonite clay, more slowly, in a manner where the effect may be more gradual and not readily apparent during construction. The impermeability of slurry walls relies to a considerable degree on the swelling properties of bentonite. Contaminants which reduce or restrict bentonite swelling may increase permeability but also can damage the self-healing properties of bentonite.

Finally, contaminants can effect not only construction practice and bentonite behavior, but also the properties of the backfill. The slurry cutoff wall backfill may lose plasticity, shrink, experience weight changes, dissolve, or petrify in response to leachates all of which can affect the slurry cutoff walls' performance. Mixtures which use cementacious ingredients (i.e. cement and fly ash) require additional considerations. The more complex the blend of materials in the slurry wall (e.g. plastic concrete > cement-bentonite > soil-bentonite) the more critical the need for examining properties of the backfill other than hydraulic conductivity as they relate to compatibility.

The system used to enact and direct the testing program is critical to successful implementation as well as the timely completion of the project. By testing the materials systematically, under worst-case scenarios, the program quickly becomes focused on workable solutions. Relatively large numbers of simple and rapid tests can be performed to eliminate borderline materials.

INDICATOR TESTS FOR COMPATIBILITY

Various indicator tests have been proposed to investigate the effect of contaminants on slurry cutoff wall materials; but to date, there is limited understanding of their applicability and even less experience to document the success of one method over another. The basis for these tests was previously developed by the petroleum, well drilling, and geotechnical disciplines. These are relatively simple tests which rely on observations and comparative results. In general, comparisons are made between performance or observations with tap water as a control (or 0.005 N $CaSO_4$) compared to a leachate. These tests are by intent worst-case models of assumed field conditions; therefore, the user must be knowledgeable to interpret and apply the results. The tests described below are those most often used by the author to evaluate compatibility.

Construction

Construction compatibility can be modeled by comparing the performance of a standard bentonite slurry in dilution with water and leachate using conventional bentonite slurry test procedures (API, RP13B-1 1990). Generally, a slurry with B/W = 5% (Bentonite/Water ratio by weight) is used and diluted 1:1 with tap water and with leachate. Depending on the application, variations in the B/W and dilution ratios may be appropriate. Because of the uncertainty in interpreting test results, it is often best to run a suite of tests. The usual tests include:

- relative filtrate loss (D'Appolonia, 1980),
- viscosity by rotational viscometer (McCandless and Bodocsi 1988), and
- sedimentation (Ryan 1987; Bowders 1985).

These tests generally give a gross indication of the expected performance of the bentonite slurry during construction and generally require only a few hours or days to perform.

The filtrate lost test is performed by pressurizing a chamber filled with slurry until a cake of pure bentonite (filter cake) is formed. The volume of water which flows out of the cake during the 30 minute long test is called the filtrate. Trench stability is dependent on a low filtrate. A second and longer test of two identical filter cakes can be performed by permeating the filter cakes with leachate and water. A ratio of flow rate with water and leachate is calculated. Generally, a rate which exceeds two indicates an incompatibility. See Fig. 1.

Similarly, a change in viscosity as measured by a rotational viscometer, may indicate the potential for construction difficulties. Identical slurries are made and then diluted with water and leachate. The viscosity of each diluted slurry is tested and compared. Changes in viscosity can be subject to various interpretations. A decrease in viscosity may result from flocculation or from a beneficial thinning of the slurry. Increases in viscosity can be the result of a viscose contaminant (e.g. petroleum) which may have no real effect on compatibility.

The sedimentation test has been used to model the construction process when the slurry is used to support the trench walls. Two identical bentonite slurries are diluted with leachate and water and observed. In this test, it is often informative to use a variety of B/W ratios for the slurry prior to dilution with the leachate because sedimentation or flocculation may be controlled to some extent by using a thicker (higher B/W) slurry or additives. Evidence of flocculation is by observation of the slurry in glass cylinders usually over a period of days.

In all of the above tests, the user must balance workability constraints (primarily viscosity and filtrate loss) with the need to address compatibility. These needs may conflict and require new materials or slurry additives to achieve the desired result.

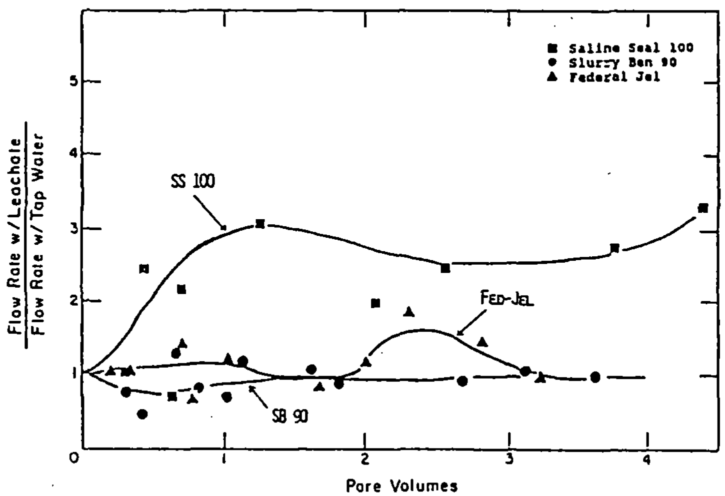

Fig. 1: Relative filtrate loss test using three bentonite clays with a landfill leachate.

Commercial Clay

Direct observations of the commercial clay product (bentonite, attapulgite, etc.) in contact with the leachate may also be used to indicate compatibility. These tests generally require a few days to complete. Again, multiple tests are used and includes:

- chemical desiccation (Alther et al. 1985), and
- free swell (McCandless and Bodocsi 1988).

These tests tend to model the most severe exposure and must be considered with some caveats. The chemical desiccation test is the drying of the bentonite slurry in contact with the leachate on a glass plate. The same standard slurry and dilution described above are used. Often severe cracking, chemical reactions, or dissolution of the clay particles can be observed. See Fig. 2. The clay is prehydrated in this test and then air dried which may be analogous to the field situation near the water table. The desiccation pattern of all clays are not identical. Some clays (e.g. sepiolite) appear unsuitable even when tested with tap water.

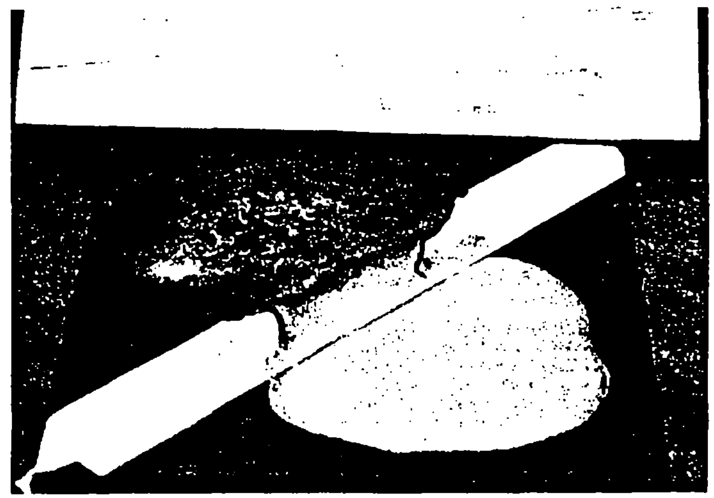

Fig. 2: Chemical desiccation test. Sample on left with leachate. Sample on right with water.

The free swell test has been used to investigate compatibility but is limited in its application since the bentonite is not prehydrated. In this test, dry bentonite particles are sprinkled into a graduated cylinder filled with water or leachate. If the bentonite does not swell, an incompatible result is indicated. In general, there is no field situation analogous to this test.

These two tests can often be used to confirm results obtained from the construction compatibility testing. The appearance of the bentonite filter cake from the filtrate loss test can be compared to the appearance of the desiccation test. Prehydrated bentonite in the sedimentation test can be compared to results from the free swell test. It is not uncommon to have apparently contradictory results.

Backfill

The slurry wall backfill material can be tested for compatibility using procedures which test the stability of the material when in contact with the leachate. Modified versions of ASTM standard tests can be used as follows:

- immersion test (ASTM Annual Book of Standards, C-267, 1991),
- fixed-wall test (ASTM D-2434, 1991), and
- plasticity (ASTM D-4318, 1991; Bowders 1985).

These tests usually require a week to a few months to complete, although typically much less time than the flexible wall test. Experience has shown that indications of incompatibility with these tests usually occurs quite early in the procedure, thereby reducing the overall testing schedule.

With cement-bentonite (CB), soil-cement (SC), and plastic concrete mixtures, a modified version of ASTM C-267, Chemical Resistance of Mortars, Grouts, and Monolithic Surfacings, can be used to investigate the physical stability of the slurry wall material. This is an immersion test where the weight and strength of the sample is measured over time in response to immersion in a leachate, as compared to immersion in water. Observations of the samples may give dramatic evidence of incompatibility. See Fig. 3. While immersion may model some conditions below the water table, only materials with a minimum unconfined strength (approximately 200 kPa) are applicable since slaking with water can produce similar weight changes in softer materials.

Soil-bentonite and other soft slurry wall materials may be tested in the fixed wall permeability cell to determine compatibility. The hydraulic conductivity developed in these tests is often of secondary importance, what is gained are observations of the potential of the material to swell, shrink, or chemically react with the leachate (Anderson et al. 1985). Since limited (or uncontrolled) effective stress is imposed, gross changes in the sample are possible which may not be possible with flexible wall permeability tests. The author has observed cases where the reaction to the leachate was so severe the sample foamed and then petrified (turned to stone), whereas no similar effect was observed in a flexible wall test. Other important physical characteristics such as resistance to high hydraulic gradients may be observed.

Replacement of pore water with leachate can change the plasticity of the backfill and therefore, hydraulic conductivity. This test works best with soil-bentonite in accordance with a modified ASTM D-4318, Liquid Limit, Plastic Limit and Plasticity Index of Soils. The user must take care to avoid imposing artificially induced effects as a result of drying. In general, the materials are slowly air dried and rewetted with tap water and contaminated groundwater and the results compared. Some mixtures can lose considerable plasticity yet retain a low permeability.

Fig. 3: Immersion test with soil-cement sample soaked in corrosive groundwater.

It has been the author's tactic to use these tests in approximately the sequence described above, using incompatible results from earlier tests, to guide in the elimination of materials with a low probability of success. The testing program usually culminates with a limited number of flexible wall permeability tests to document long-term hydraulic conductivity in the presence of the leachate. With a knowledgeable selection of tests, materials and additives based on the indicator tests, the final flexible wall tests are nearly always successful.

CASE STUDIES

The projects described below have been selected from the author's files of over a hundred successful projects. These case studies have been selected because they represent projects where an incompatibility was discovered and/or alternate materials were used to provide a suitable solution. The author has, by intent, limited the discussion to the facts of the case related to the determination of incompatibility and the finding of an alternate solution.

Case Study No. 1: Southern Wisconsin Landfill

An operating sanitary landfill was closing a formerly uncontrolled landfill cell which had received hazardous wastes. Physical and hydraulic isolation of the cell was necessary to comply with regulatory directives to protect the environment. Closure of the cell included a RCRA cap, groundwater collection trench and soil-bentonite slurry cutoff wall.

Leachate from the landfill was generally characterized by a black color and pungent odor with high chloride (about 500 mg/l) and sulfate (about 10 mg/l) contents. The groundwater plume emanating from the site was found to contain toxic levels of organic chemicals including vinyl chloride. Contaminant levels were high enough that reuse of trench spoil in the soil-bentonite backfill was not permitted. Compatibility testing of the soil-bentonite backfill began with the development of a bentonite slurry for trenching. Three products were tested; two premium grade, sodium (API 13A) bentonites and one "contaminant-resistant," SS100 bentonite. A stable slurry with a B/W = 5% was produced from all three bentonites with a viscosity (Marsh Funnel) of 40 to 50 seconds without the use of additives.

Relative filtrate loss tests using the leachate and tap water are shown in Fig. 1. It was observed that the SS100 bentonite permeated with the leachate produced a relative filtrate loss three times greater than with tap water and much higher than either of the premium bentonites. In the desiccation test, a pattern of small cracks was observed with the SS100 which was not present in tests of the other bentonites. Finally, a sedimentation test of the bentonites was performed as shown in Fig. 4. In this test, all three bentonites performed similarly.

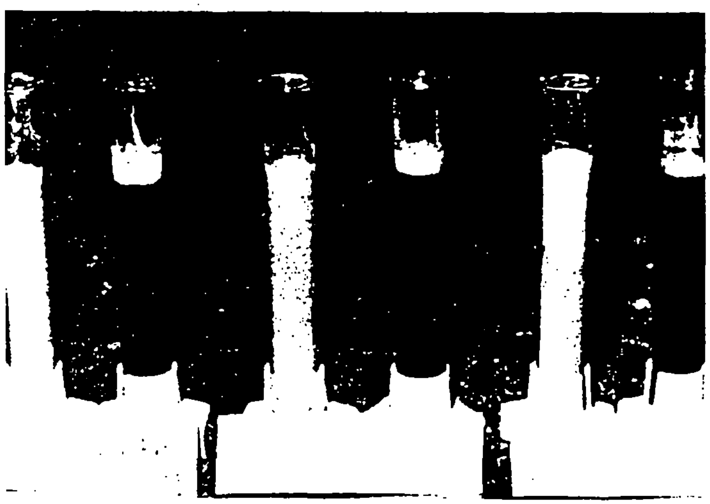

Fig. 4: Sedimentation test with three bentonite clay mixed with landfill leachate and with water.

Based on these results, SS100 bentonite was excluded from further consideration. The remainder of the test program, including hydraulic conductivity testing, proceeded successfully.

A 1200 meter (4000 ft) long by 10 meters (35 ft) deep slurry cutoff wall was installed which has, since 1987, prevented the further contamination of the area. Tests show that the slurry cutoff wall was effective and the vinyl chloride plume dissipated.

Case Study No. 2: Eastern Michigan Chemical Facility

A chemical plant was operating a system of treatment lagoons which abutted a former brine production area separated by a relatively narrow earthen dike. Closure of the brine ponds without disturbance to the treatment lagoons, using a slurry cutoff wall, was the aim of the project. The brine contained high levels of metals including calcium (8.3%), magnesium (0.60%), and sodium (1.61%). Total dissolved solids in the leachate was 25 to 30% and the density of the brine was 1.04, gm/cc.

Implementation of the project was complicated by at least three compatibility concerns:

1. brine is known to flocculate bentonite slurry,
2. chemicals in the treatment lagoons could have an unknown effect on the slurry wall, and
3. the dike was unstable (safety factor < 1.0) and required reinforcing.

The compatibility testing for this project began with the selection of an alternate clay to replace bentonite. Testing of premium bentonite, "saline-resistant" bentonite and attapulgite was conducted as shown in Fig. 5. In this case, attapulgite, a nonswelling montmorilite clay (Tobin and Wild 1986) was found to be most effective. In addition, attapulgite could be mixed with brine water for the trenching slurry. Using attapulgite with the brine water and wastewater also produced successful results in the desiccation and sedimentation tests.

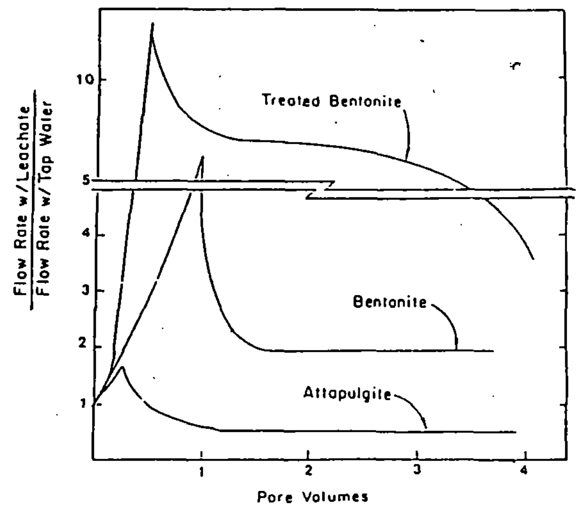

Fig. 5: Relative filtrate loss test using three commercial clays with brine water leachate.

Stabilization of the dike required a cementacious backfill which would reinforce the dike and increase the factor of safety against sliding. Cement-attapulgite (a variation of cement-bentonite self-hardening slurry) and plastic concrete mixtures were te[illegible] with permeabilities less than 1 x 10^{-6} cm/sec. Results of the unconfined compressive strength tests are shown in Fig. 6. Immersion tests and long-term permeability tests with the leachate were performed which demonstrated the compatibility of the cement-attapulgite with the brine water.

Based on the results described above, a 700 m (2,000 ft) long cement-attapulgite slurry trench about 10 m (30 ft) deep was constructed through the center of the dike. Brine water was used as the mix water for the slurry. Since 1988, the project has served to separate the wastewater pond and the brine pond. The stability of the dike has been ensured by the use of the cement-attapulgite.

Case Study No. 3: Upstate New York Lagoon Closure

A former mine and processing plant produced two byproducts which were co-mingled in a single earthen-lined lagoon. One byproduct, semet, has a pH < 0.5 and the other byproduct has a pH > 13. Storage of the two byproducts in a single lagoon did not produce neutralization and the leachates were found to be existing separately and seeping out of the lagoon into the groundwater.

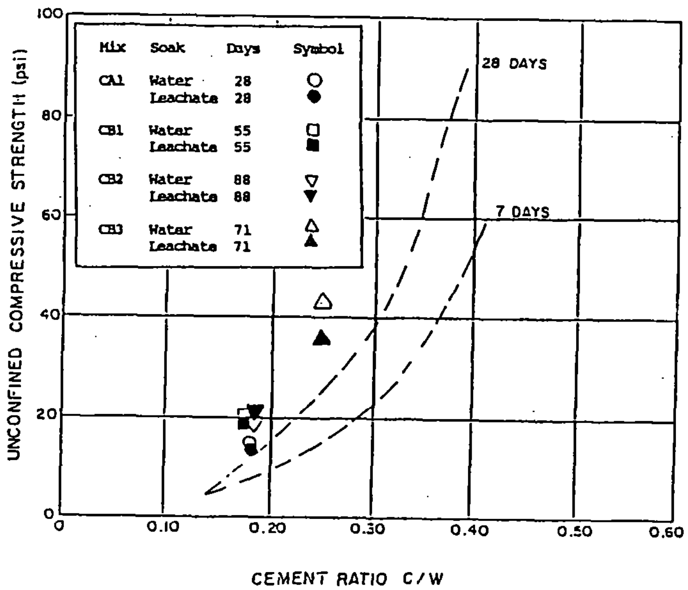

Fig. 6: Unconfounded compressive strength of cement-attapulgite immersed in water and low pH leachate. Comparative trends for Millet and Perez (Millet and Perez 1981)

At this time, one of the potential remedies to the sites is containment with a cutoff wall. The wall will be more than 30 m (100 ft) deep so deep soil mixing (DSM) and plastic concrete are considered as prime candidates for the cutoff wall. Compatibility testing for this site provides an opportunity to test the limits of the testing methods.

Testing began with separate tests of the high and low pH leachates with a variety of commercial clay products. As previously described, a step-by-step process was enacted which focused the program on the most critical compatibility challenge. The high pH leachate was compatible with all clays in the filtrate, sedimentation, and desiccation tests. Therefore, the majority of the program was focused on compatibility of materials with the low pH semet leachate. Filtrate, sedimentation, and desiccation testing proved that attapulgite was the best commercial clay to resist the semet. What remained, therefore, was to find a combination of soil and/or cement to complement the attapulgite. Initial tests with soil-attapulgite were carried out with fixed wall permeameters. The results were dramatic and unsuccessful. The leachate reacted violently with soil-attapulgite producing a gas and turning the sample into a petrified mass. Immersion tests with soil-cement-attapulgite (at relatively low total cement contents) were equally unsuccessful. As shown in Fig. 3, many of the samples dissolved in the immersion tests. Finally, cement-attapulgite blends (with relatively high total cement contents) were found which survived the immersion tests. The strength of immersed cement-attapulgite was similar to cement-bentonite mixture as shown in Fig. 6. Long-term flexible wall permeability tests confirmed the compatibility of the cement-attapulgite by the display of a stable hydraulic conductivity over three pore volumes of flow. Dissection of a cement-attapulgite sample after permeation is shown in Fig. 7 to illustrate the success of the compatible mixture.

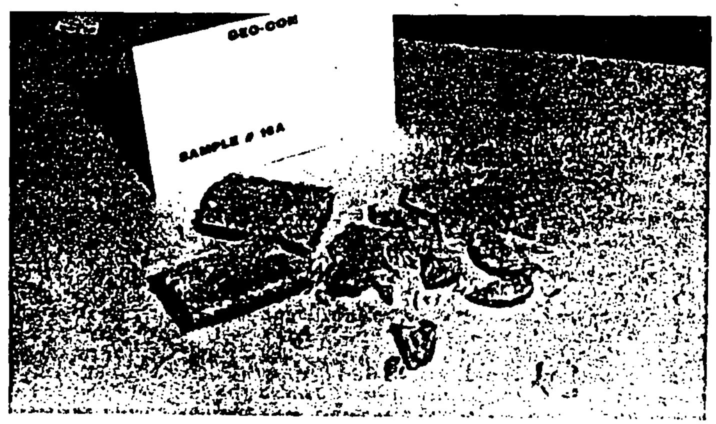

Fig. 7: Dissected cement-attapulgite sample after permeation by low pH leachate for three pore volumes.

Case Study No. 4: Former Industrial Site in Vancouver, B.C.

A site which borders the bay in the center of Vancouver had been used since the city's founding for a variety of industrial purposes including coal gasification, wood treatment, and fuel storage. A variety of toxins were found in the soils and groundwater including cyanide (10 ppm), hydrocarbons (100 ppm), pentachlorophenol (20 ppm), arsenic (1 ppm), lead (4 ppm), and zinc (6 ppm). In order to reclaim and develop the site, A DSM and jet grout wall was constructed to contain the contaminants. Development of the site requires excavation of an area of significant contamination and eventually building foundations; therefore, the cutoff walls were specified to have an unconfined compressive strength of up to 1.4 MPa (200 psi) as well as a hydraulic conductivity less than 10^{-6} cm/sec.

Due to the structural requirements and the availability of resources, the testing program focused on soil-cement blends which used a grout composed of Canadian calcium bentonite, Wyoming sodium bentonite, gypsum, fly ash, and cement. The use of gypsum was selected to provide improved strength with reduced permeability. Calcium bentonite is a low swelling bentonite clay which provides stability to the grout and reduces permeability. Concerns about the use of these innovative materials, as well as requirements for compatibility, resulted in an extensive testing program.

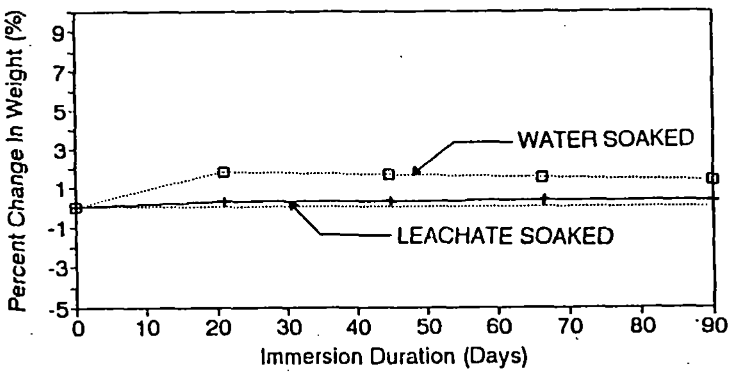

Fig. 8: Immersion test result of DSM sample in water and hazardous leachate.

Testing of the bentonite resulted in the finding that at least three times as much Canadian calcium bentonite (B/W = 15%) as Wyoming sodium bentonite was necessary to produce a workable slurry. The addition of cement and fly ash to this slurry required thinners including both phosphate and lignosulfate based products.

The addition of gypsum provided beneficial thinning of the grout; and, therefore, the use of a relatively dense grout with no loss in workability. Once blended into the mix, the gypsum becomes a part of the cement matrix. No dissolution or other detrimental effects were noted with the use of gypsum.

Compatibility testing focused on immersion testing and flexible wall permeability testing of the soil-cement. Immersion tests were conducted for up to 90 days in the leachate. The immersed samples appeared identical in water and leachate with an average weight change of less than 1%. The majority of any weight change was usually discovered within the first 28 days of immersion. See Fig. 8. Hydraulic conductivity tests on the hardened soil-cement confirmed the long term stability of the materials.

The cutoff wall was constructed in the summer of 1992. Each type of cutoff wall and grout mixture was subjected to extensive field testing including test sections which were excavated and examined. In total, over 600 m (2,000 ft) of cutoff wall were installed up to 16 m (50 ft) deep. Insitu testing and monitoring to date has shown the cutoff wall to be highly effective.

CONCLUSIONS

A systematic approach to compatibility testing includes indicator tests along with permeability tests. Compatibility testing using indicator tests provides a relatively rapid and rational method for predetermining the compatibility of slurry cutoff wall materials with contaminated groundwater. Not all indicator tests are applicable on every project. Furthermore, some tests model situations which are impossible on some sites. The tests are relatively simple and rapid, but the application of the results to real remediation projects requires the expertise of a knowledgeable engineer and specialty contractor with experience in the materials selected for installation.

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THE COMPATIBILITY OF SLURRY CUTOFF WALL MATERIALS WITH CONTAMINATED GROUNDWATER

Steven R. Day, Geo-Con, Inc., Pittsburgh, Pennsylvania, U.S.A.

REFERENCE: Slurry cutoff walls are frequently relied upon to block groundwater flows from toxic waste sites and landfills. The long-term effectiveness of slurry cutoff wall materials is critical to the successful containment of these facilities and the protection of groundwater resources. A variety of laboratory indicator tests have been attempted by engineers and academia to make compatibility determinations but at present there has been little published experience to show which tests produce meaningful results and how these tests can be used to demonstrate compatibility.

Hydraulic conductivity is a useful measure of chemical/soil compatibility but permeability tests alone cannot assure the long-term stability of a slurry cutoff wall. A suite of indicator tests are used where the leachate and the proposed materials are combined and tested in immersion, desiccation, sedimentation, and other modes. Each indicator test attempts to model a different scenario of the slurry cutoff wall installation and operation.

This paper presents the experience of a specialty contractor from a number of projects, where an incompatibility was discovered and alternate materials were used to find a successful solution. Monitoring results from these sites has proven the effectiveness of the chosen solution. The laboratory test methods described are relatively simple and rely on worst case scenarios, performed in a step-by-step process, which culminates with flexible wall permeability tests. Based on the methods described and the results from successful projects where these methods were used, engineers, owners and the public may better rely on long-term slurry cutoff wall performance with an increased level of confidence.

Key Words: attapulgite, bentonite, compatibility, containment, deep soil mixing, hydraulic conductivity, jet grouting, slurry cutoff wall

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Hazardous Waste Engineering
Research Laboratory
Cincinnati OH 45268

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EPA

Project Summary

Investigation of Slurry Cutoff Wall Design and Construction Methods for Containing Hazardous Wastes

Richard M. McCandless and Andrew Bodocsi

Specific technical design standards for soil-bentonite slurry trench cutoff walls used to isolate hazardous wastes have not been established. A review of current design and construction methods was performed for summarizing current engineering practice, identifying areas of technical debate, and initiating necessary research to promote the development of rational standards. The review of current methods was followed by laboratory studies using specialized test equipment to study model cutoff walls.

An instrumented slurry test column was developed and used to investigate the hydraulic characteristics and importance of bentonite slurry seals formed on the walls of the cutoff trench during construction. Testing involved the penetration of a 5% bentonite: water slurry into two different sands, the formation of a different type of slurry seal in each case, and the measurement of their hydraulic conductivities based upon the time-rate of flow and the measurement of internal pore pressure conditions. The effectiveness of different slurry seals varied greatly depending upon the degree of filtration of hydrated bentonite particles during slurry penetration into granular soils. In all cases, however, the effectiveness of the seals alone (ignoring the contribution of the soil-bentonite backfill) was very low, suggesting that they cannot be relied upon to offset the effects of latent defects in the backfill, and that the current practice of disregarding the slurry seal in cutoff wall design should not be changed.

Laboratory testing also involved an instrumented slurry wall tank capable of accommodating 508 mm (20 inches) diameter, 101.6 mm (4 inches) thick model cutoff walls. The tank was used to evaluate the effects of overburden pressure (vertical consolidation) and hydraulic gradient (horizontal consolidation), and to evaluate the potential for self-remediation of hydraulic defects ("windows" through the barrier) via in situ consolidation of the soil-bentonite backfill. Various models were permeated with water under varying hydraulic gradients and vertical surcharge pressures. The average equilibrium hydraulic conductivity of the models was measured under each set of conditions. Results demonstrated that both overburden pressure and hydraulic gradient have significant and comparable effects on the average conductivity of the wall. Moreover, water content, unit weight, and vane shear strength data measured on samples of the soil-bentonite backfill after the test clearly indicated that effective overburden stress decreased with increasing depth in the model, most likely due to friction between the backfill and sand in which the model was constructed.

Another model wall was intentionally breached by two slot-like "windows" representing small pockets of entrapped bentonite slurry in the backfill immediately after construction. By incrementally increasing surcharge pressure it was possible to "heal" the windows as evidenced by a return to the predetermined baseline hydraulic conductivity of the wall. This suggests that in situ

consolidation of the backfill may help to eliminate some types of as-built hydraulic defects or micro-cracks within the backfill resulting from long-term chemical degradation.

This Project Summary was developed by EPA's Hazardous Waste Engineering Research Laboratory, Cincinnati, OH, to announce key findings of the research project that is fully documented in a separate report of the same title (see Project Report ordering information at back).

Introduction

Slurry trench cutoff walls were first used in the United States in the early 1940's. Since that time, their use has become more widespread and now includes application as hydraulic barriers to control the movement of contaminated groundwater from hazardous waste disposal sites. Specific technical design standards for slurry trench cutoff walls (also known as soil-bentonite walls) have not been established. Each application is unique and requires site-specific engineering evaluation. Nevertheless, the current state-of-the-art involves fundamental concepts, performance criteria, and methods common to all applications. The purpose of this project is threefold:

- to compile information on current design and construction methods
- to identify specific research needs to promote the development of rational standards
- to perform initial research in selected areas of need

The first phase of the project involved review of published literature on slurry wall technology, interviews with owners, engineering consultants and construction contractors, and a general assessment of methods and research needs. Based upon these findings, two subsequent research phases emphasized laboratory model studies of slurry seals formed on the walls of a cutoff trench during construction and small model cutoff walls incorporating both slurry seals and a standard soil-bentonite backfill.

Specific objectives of the laboratory studies were to determine or evaluate:

- the depth of penetration of slurry or filtered slurry into typical granular soils
- the hydraulic conductivity of various types of seals derived from slurry penetration and slurry filtration during penetration into typical granular soils
- the stability of the seals (described above) after initial development
- in situ consolidation and the effect of surcharge loading and hydraulic gradient on soil-bentonite hydraulic conductivity
- the feasibility of "window" closure within a soil-bentonite wall due to overburden consolidation pressures.

Current Methods

The initial phase of this study involved a survey of current design and construction methods which form the basis of present slurry cutoff wall technology. The survey involved review of published literature on the subject, interviews with selected vendors and professional practitioners specializing in slurry wall applications, and visits to three slurry wall construction sites. the report does not attempt to quantify the variability in present methods but simply documents the range of philosophy and current practice in the areas of Design, Specification, Construction and QA/QC. The specific considerations that are least standardized, and therefore most variable, in each subject area are summarized below:

Design	• soil-bentonite mix design • method of hydraulic conductivity testing • bentonite type • bentonite content in the backfill • the use of contaminated trench spoils in the backfill
Specification	• performance type or materials and methods type
Construction	• backfill mixing/handling techniques • backfill placement method • equipment type • personnel - level of experience
QA/QC	• verification of trench depth, width and continuity • personnel - level of training/experience • responsibility - contractor, consultant or owner? • frequency and manner of backfill testing

Laboratory Investigations Procedures

Slurry Seals

An instrumented slurry test column was developed to study various bentonite slurry seals formed on the walls of the cutoff trench during construction. The system consists of an acrylic column equipped with probes to measure in situ pore pressure after the formation of a slurry seal in different sands. Spring-suspended inflow (head) and outflow (tail) permeant reservoirs were employed to achieve constant-head test conditions. A schematic of the system is shown in Figure 1. Pore pressures were monitored during permeation to produce data on the depth of the slurry penetration, the hydraulic conductivity of the overall seal, and changes in these features as a function of time.

A clean fine sand identified herein as the "+200 sand" (retained on the no. 200 sieve) was used to study the surface filtration (filter cake) type of slurry seal in the slurry test column. This sand is predominantly fine, of roughly uniform size (no. 40 to no. 50 sieve size), with about 25 percent medium sand by weight. A clean medium to coarse sand was used to investigate deep filtration and rheological blockage seals. The gradation comprised roughly 75% medium sand and 25% coarse sand, with all material being retained on the no. 40 sieve ("+40 sand").

All tests involved slurry seals derived from the penetration of a standard 5 percent bentonite: water slurry (weight: volume basis). Slurry was driven into the test sands under controlled pressure (seal formation pressure) for a standard period of five hours. Seals formed in this manner were then permeated by water under variable hydraulic pressures sometimes different than the seal formation pressure. Testing comprised both saturated and unsaturated cases to model conditions below and above the groundwater table, respectively.

In all cases, hydraulic conductivity data were calculated from several parameters measured during the test. These parameters included the pressure differential between any two pore pressure probes, the physical distance between the probes, and the volume flow-rate through the sample (discharge per unit time).

Figure 2 shows typical pore pressure distributions during steady flow for the +40 and +200 sands under roughly equivalent hydraulic gradients. In each case, the data demonstrate a nearly

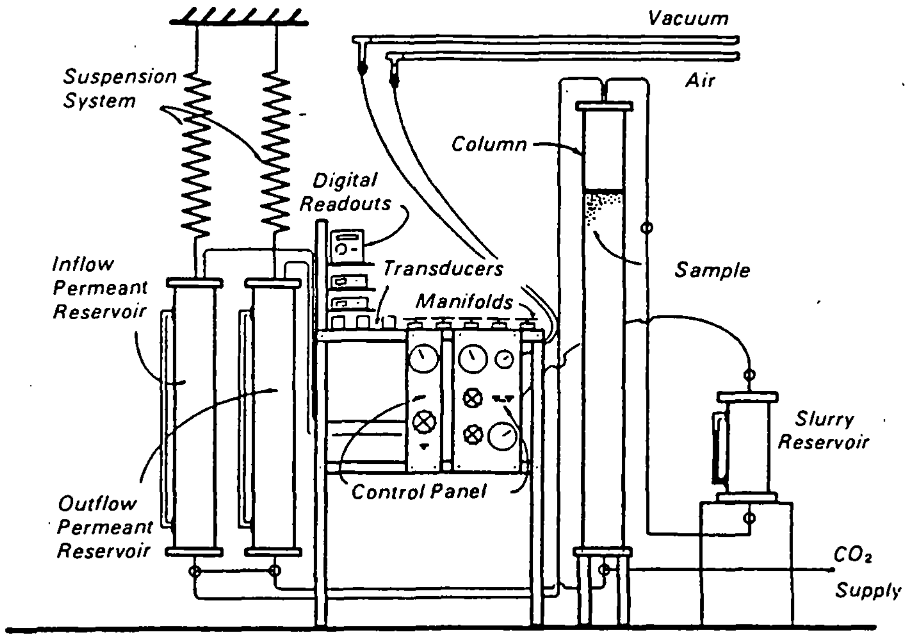

Figure 1. Schematic of the slurry test column system.

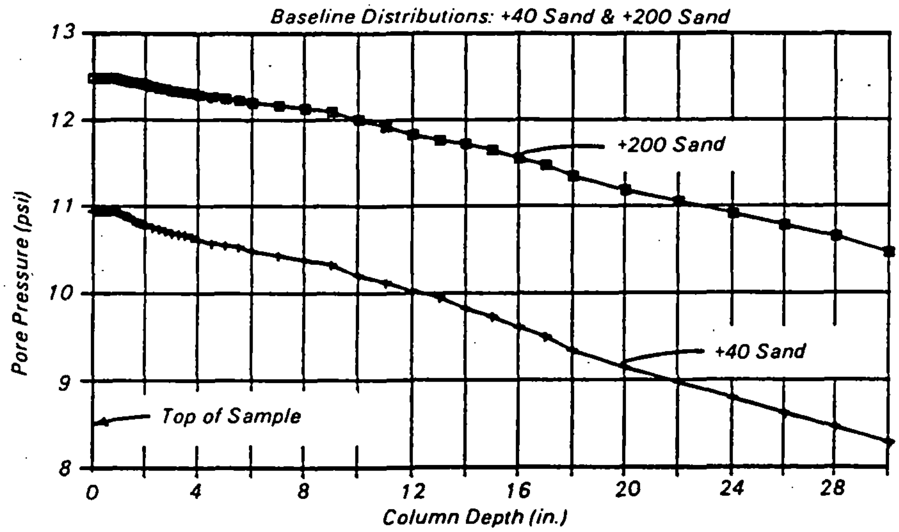

Figure 2. Typical baseline (no slurry seal) pore pressure distribution for fine (+200) and medium to coarse (+40) sands used in this study.

constant rate of head loss through the sample prior to the introduction of slurry. After development of a slurry seal, the steady-state pore pressure distributions for the +40 and +200 sands were as shown in Figure 3a and 3b, respectively. Data such as these were used to define the location, thickness, and hydraulic gradient across the seals, from which their hydraulic conductivities were computed.

Results

Slurry Seals

Numerous tests were performed on both the +40 and +200 sands at seal formation and permeation pressures ranging from 9.3 kPa (1.35 psi) to 68.95 kPa (10.0 psi). Of these, only two tests of the +40 sand and five tests of the +200 sand produced useable data. In most other tests the slurry seals were breached by the combined effects of cracking and erosion (piping) from beneath. The cause is believed to be related to minor pressure fluctuations within the system in response to temperature changes and/or supply pressure changes from day to night and vice-versa. These pressure fluctuations would cause differential expansion/contraction between the acrylic column and the sand. Such disturbance would cause micro-cracks in the seal followed by progressive widening of the cracks via erosion. It was possible, however, to generate comparative *initial* permeability data for the seven tests described above, and to compute the "breakthrough time" (time for the first drop of permeant to pass through the cutoff wall barrier) for the two types of slurry seals.

Figure 4 is a schematic of two typical soil-bentonite walls, showing the expected zone of slurry penetration and seal formation in the +40 and +200 sands. Deep slurry penetration accompanied by rheological blockage occurs in the +40 sand, whereas a surface filtration seal is shown for the +200 sand. In both schematics, the soil-bentonite backfill is assumed to be the same, having a hydraulic conductivity of 1.0×10^{-7} cm/sec. The depth of slurry penetration and the hydraulic conductivity of the seal in each case are based upon results obtained using the slurry test column.

Assuming the same in-service head differential across each barrier and steady flow according to Darcy's law, it was determined that the effectiveness of the wall in the +40 sand based upon a breakthrough criterion would be about three times as much as that of a similar wall constructed in a deposit of +200 sand (93.5 years vs. 31.0 years). Moreover, the breakthrough times of the two slurry seals alone (no soil-bentonite backfill) was determined to be on the order of two weeks or less.

Procedures

Model Cutoff Walls

The slurry wall tank constructed for this study accommodates circular cutoff walls roughly 559 mm (22 inches) in height, 102 to 152 mm (4 to 6 inches) thick, and up to 610 mm (24 inches) in diameter. The tank is of stainless steel construction and employs a pneumatic bladder system to vertically confine and consolidate the model wall during permeation in the horizontal direction. A schematic of the system is shown as Figure 5.

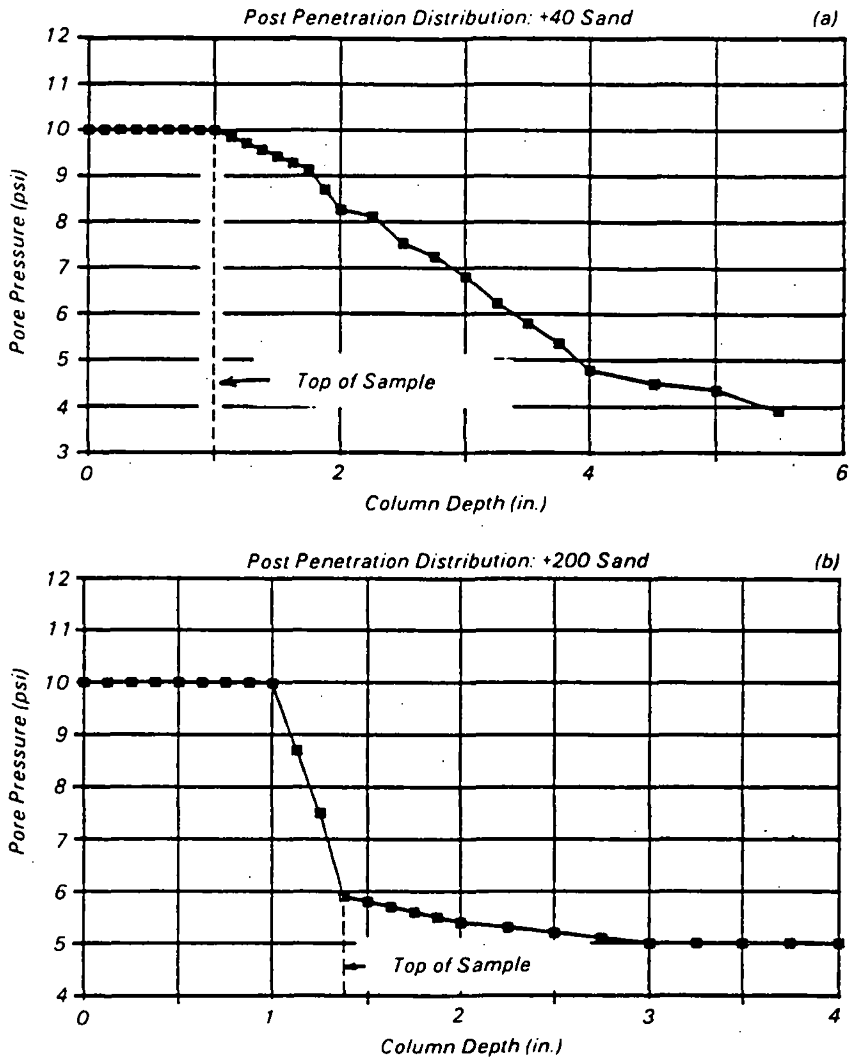

Figure 3. Typical initial pore pressure distributions after formation of slurry seals in the (a) +40 and (b) +200 sands.

The model walls were constructed between two concentric PVC (polyvinyl chloride) slip forms representing the walls of a circular cutoff trench. The forms were positioned in the tank and backfilled with clean fine sand in 102 mm (4 inch) lifts creating an empty 102 mm (4 inch) wide annular space between the forms. This space was then filled with a 5% bentonite:water slurry (weight: volume basis) comprising the same bentonite used in the soil-bentonite mix. The soil-bentonite backfilling operation varied slightly for different models but generally involved raising both forms about 102 mm (4 inches), allowing the bentonite: water slurry to penetrate the sand and form a surface filtration slurry seal, and then backfilling with soil-bentonite using a pressurized tremie pipe. This general procedure was repeated until the surface of the model wall was level with the surface of the center core of sand (sand encircling the model wall).

After construction, the model was readied for testing by installing a combination membrane/hydraulic cutoff over its surface and positioning concentric load-bearing plates over each element of the model (core sand, soil-bentonite wall, outer ring of sand). This arrangement allowed for differential loading and consolidation of the soil-bentonite wall relative to the adjacent sand bodies.

The typical testing procedure used in evaluating the effects of overburden pressure and gradient involved saturation of the sand elements of the model, application of a selected surcharge pressure, consolidation of the soil-bentonite wall under the applied surcharge (time estimated from conventional consolidation tests performed on the backfill material), application of the design hydraulic head pressure at both the top and bottom of the saturated center core of sand (Figure 5), and the measurement of hydraulic head and volumetric inflow at prescribed time intervals.

Similar procedures were used in the construction and testing of the third model wall to evaluate the closure of artificial slot-like windows via surcharge pressure. The slots were intended to model macro-defects such as small pockets of entrapped slurry remaining after construction of the wall. Two slots approximately 7.9 mm (5/16 inch) wide by 1.6 mm (1/16 inch) high were cut into the third wall after preconsolidation under an effective overburden of 41.4 kPa (6.0 psi) as measured at the surface of the wall. The windows were positioned 180° apart at a depth of about 127 mm (5 inches) below the top of the wall. Both ends of each slot were covered with a fabric-covered wire mesh to prevent washing the core sand into the slot during permeation. The test procedure involved incremental increase of overburden (surcharge) pressure until the slots were effectively closed as evidence by a return to the predetermined baseline hydraulic conductivity of the model.

Results

Model Cutoff Walls

The testing of model slurry walls involved staged incrementation of overburden pressure and hydraulic gradient, followed by sampling and measurement of unit weight, vane shear strength and moisture content as a function of depth in the model. Three different hydraulic gradients (i = 21, 42, 83) were applied under effective overburden pressures of 41.4, 82.7 and 165.5 kPa (6, 12, 24 psi) as measured at the surface of the wall. Figure 6 presents a chronological summary of the final equilibrium conductivities measured for each set of test conditions. Initial hydraulic conductivities are represented by an open triangle and final equilibrium values by an open circle. Two incidences of hydrofracture are indicated by solid triangles.

Except for test 2(g), the data suggest a logical trend of decreasing equilibrium hydraulic conductivity as a function of either increasing surcharge pressure or increasing hydraulic gradient. The data do not, however, reflect the correct magnitude of change in hydraulic con-

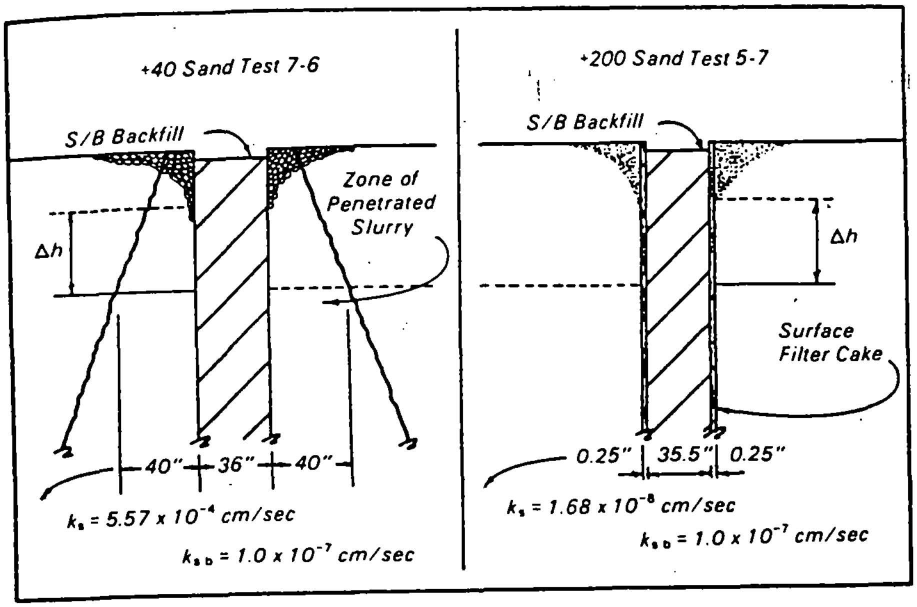

Figure 4. Idealized conditions after construction of cutoff walls in the +40 and +200 sands.

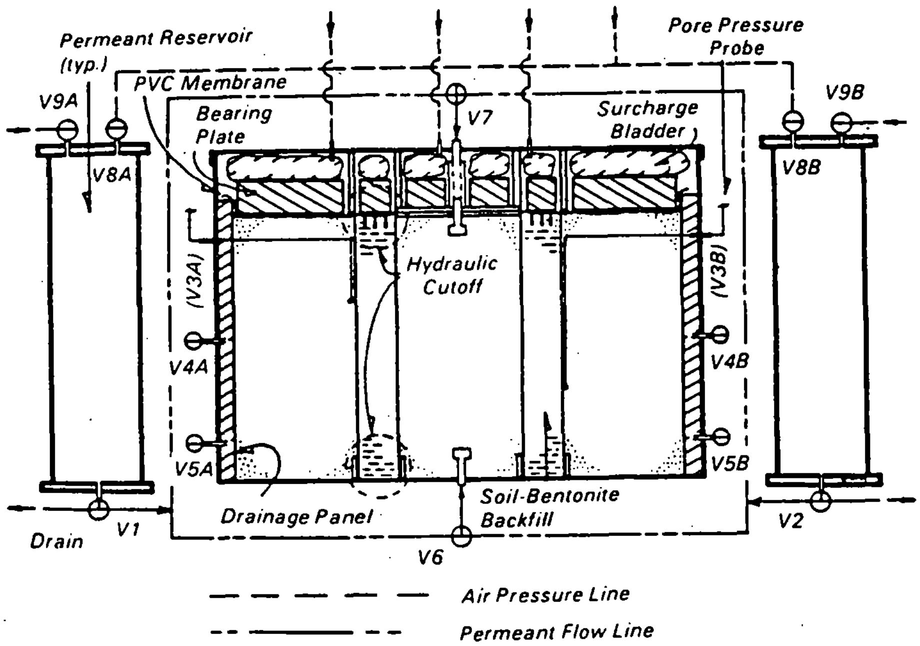

Figure 5. Schematic of the slurry wall tank system.

ductivity between successive tests. The reason is that hydrofracture permanently changed the properties of the wall, thus artifically offsetting groups of data measured after hydrofracture from other groups of data measured before hydrofracture.

After the completion of test 2(g) reported in Figure 6, the tank was opened to permit inspection of conditions and allow for sampling and testing of the backfill. Testing involved measurements of unit weight, vane shear strength and water content. Data for these parameters appear as a function of depth in Figure 7.

After sampling and inspecting of the model a new wall was constructed for the window closing test. After establishing a baseline or reference value of hydraulic conductivity, the two slot windows were formed at the locations and depths previously described. Overburden pressure was than gradually increased causing the apparent hydraulic conductivity of the model to decrease until the windows had been effectively closed as evidenced by a return to the measured baseline conductivity.

Conclusions for Slurry Seals

- For seals formed on fine sands by the surface filtration mechanism: 1) the density of a seal is proportional to the density of the sand in which the seal forms and proportional to the prevailing hydraulic head under which the seal forms, 2) the hydraulic conductivity of a seal is inversely proportional to the prevailing hydraulic head under which the seal forms and inversely proportional to the density of the sand in which the seal forms, and 3) the thickness of the seal is a function of formation time only.
- Based upon the unknown frequency of chemically induced or construction-related "windows" in a typical soil-bentonite cutoff wall, it appears that the current practice of design on the basis of the permeability of the soil-bentonite backfill alone should not be changed.

Conclusions for Model Cutoff Walls

- The average hydraulic conductivity of model cutoff walls was observed to decrease both as a function of increased overburden pressure (vertical consolidation), and increased hydraulic pressure (horizontal consolidation due to hydraulic gradient), as well as their combined effect.
- Hydrofracture, or rupture of the cutoff wall may be induced in the subsurface at locations where the hydraulic driving pressure exceeds the effective vertical overburden pressure. Although the applied surcharge pressure at the top of the wall in these cases was higher than the hydraulic pressure, it was not effective over the full depth of the wall resulting in general hydrofracture (presumably near the base of the wall).
- Density, water content and vane shear strength data measured on samples from a cutoff wall after

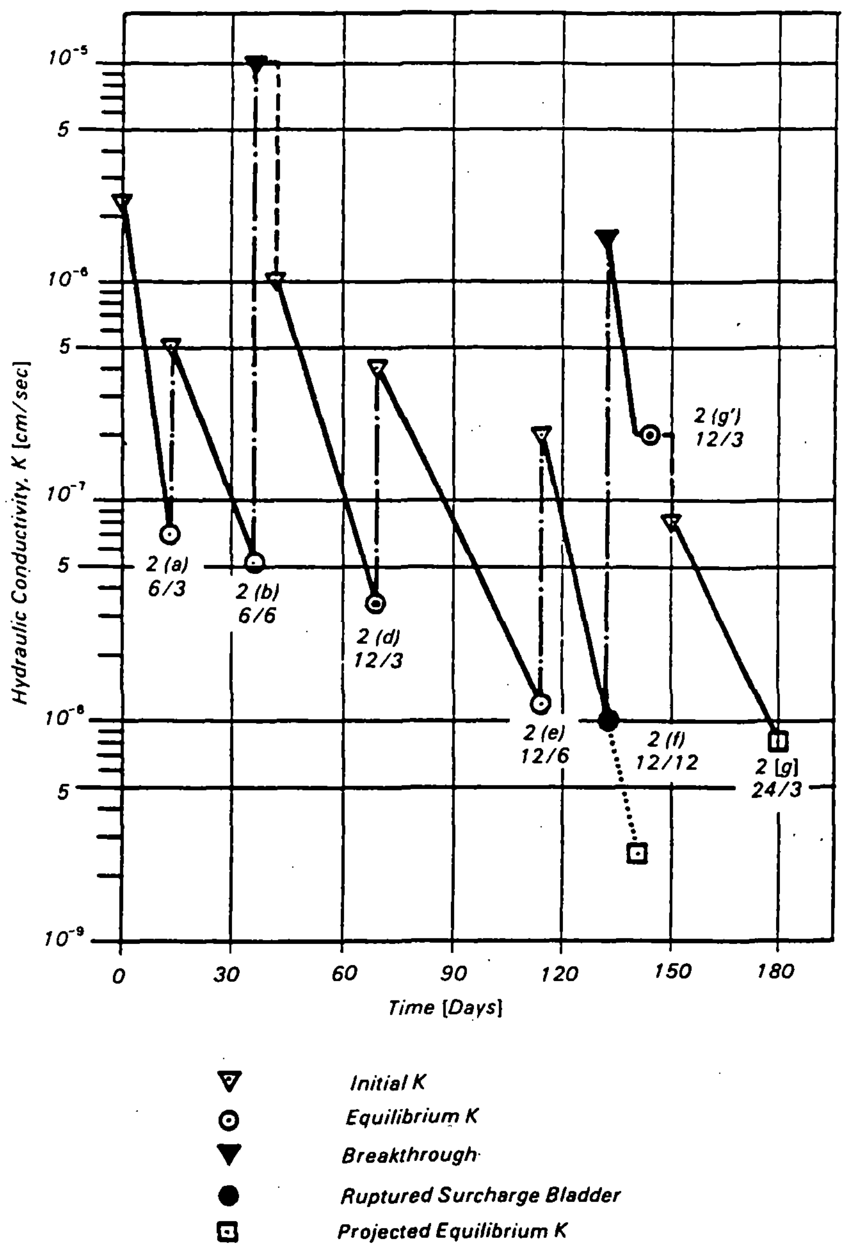

Figure 6. Chronology and results of hydraulic conductivity tests.

testing all confirm the dissipation of vertical overburden pressure with increasing depth in the model.

- The success of the window closing test suggests that the effective overburden pressure in the wall may serve to close residual slurry windows and may even close a multitude of micro shrinkage cracks that may develop in the backfill over the life of the barrier due to the effects of chemical leachates.

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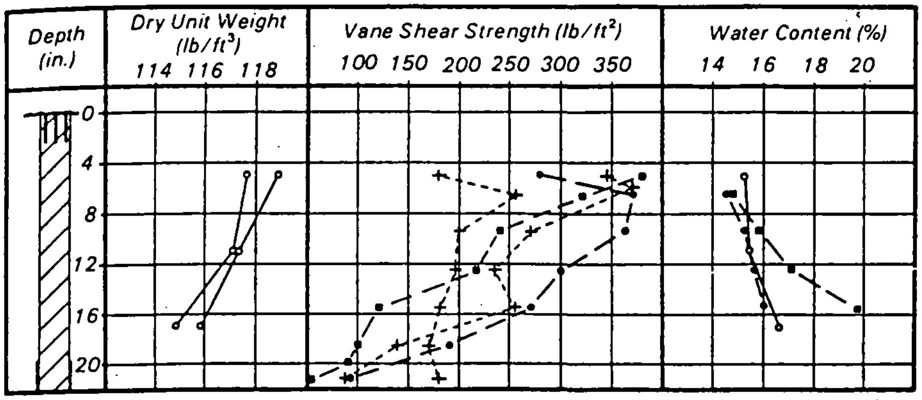

Figure 7. *Results of tests on soil-bentonite backfill after completion of hydraulic conductivity tests.*

Richard M. McCandless and Andrew Bodocsi are with the University of Cincinnati, Cincinnati, OH 45221.
***Naomi P. Barkley** is the EPA Project Officer (see below).*
The complete report, entitled "Investigation of Slurry Cutoff Wall Design and Construction Methods for Containing Hazardous Wastes," (Order No. PB 87-229 688/AS; Cost: $24.95, subject to change) will be available only from:
National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
Telephone: 703-487-4650
The EPA Project Officer can be contacted at:
Hazardous Waste Engineering Research Laboratory
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Cincinnati, OH 45268

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Chapter 7

Vertical Cutoff Walls

7.1 Introduction

Situations occasionally arise in which it is necessary or desirable to restrict horizontal movement of liquids with vertical cutoff walls. Examples of the use of vertical cutoff walls include the following:

1. Control of ground water seepage into an excavated disposal cell to maintain stable side slopes or to limit the amount of water that must be pumped from the excavation during construction (Fig. 7.1).

2. Control of horizontal ground water flow into buried wastes at older waste disposal sites that do not contain a liner (Fig. 7.2).

3. Provide a "seal" into an aquitard (low-permeability stratum), thus "encapsulating" the waste to limit inward movement of clean ground water in areas where ground water is being pumped out and treated (Fig. 7.3).

4. Long-term barrier to impede contaminant transport (Fig. 7.4).

Vertical walls are also sometimes used to provide drainage. Drainage applications are discussed in Chapters 5 and 6.

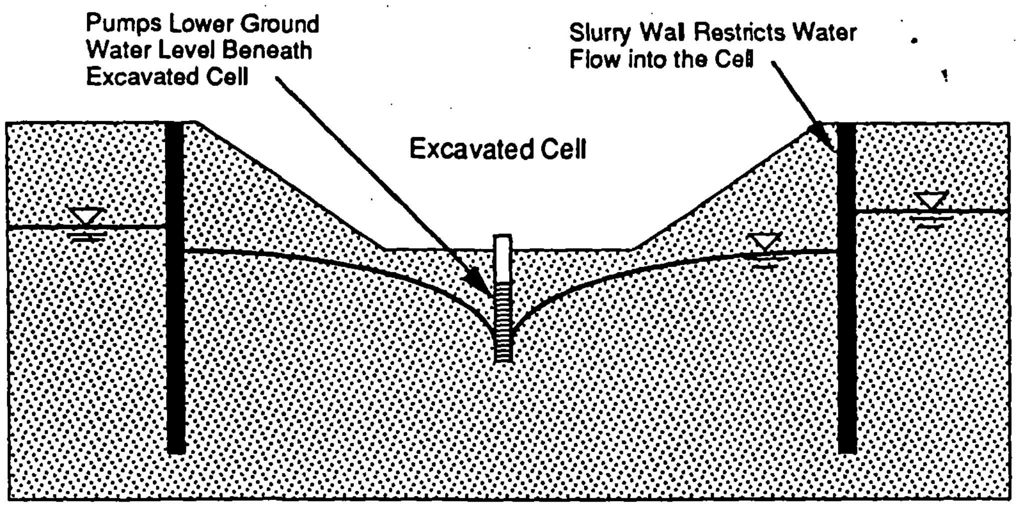

Figure 7.1 - Example of Vertical Cutoff Wall to Limit Flow of Ground Water into Excavation.

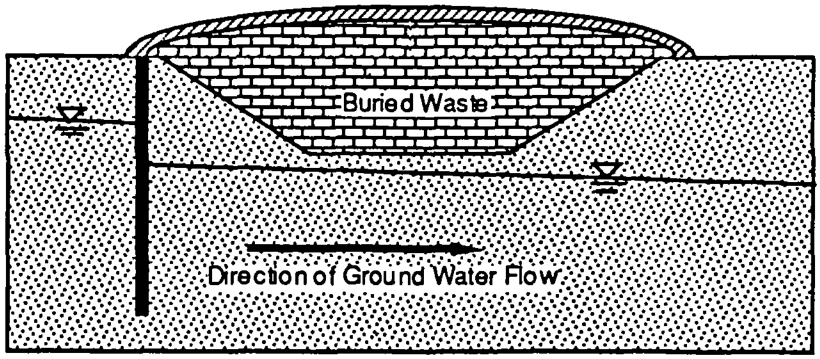

Figure 7.2 - Example of Vertical Cutoff Wall to Limit Flow of Ground Water through Buried Waste.

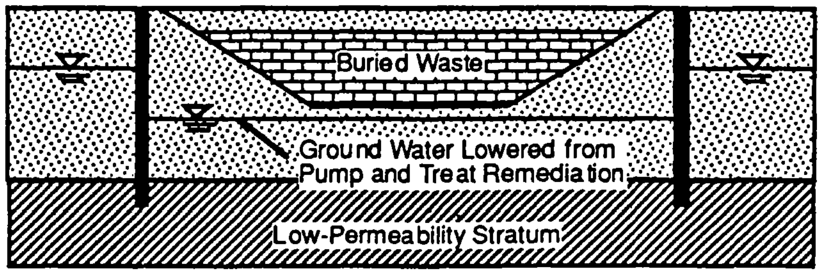

Figure 7.3 - Example of Vertical Cutoff Wall to Restrict Inward Migration of Ground Water.

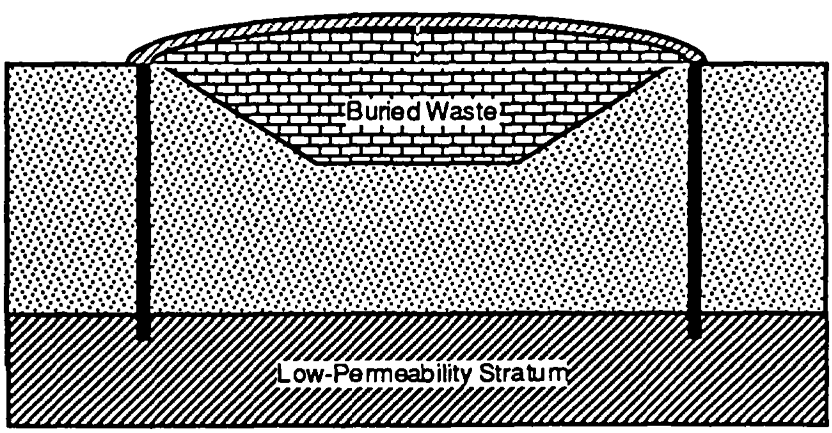

Figure 7.4 - Example of Vertical Cutoff Wall to Limit Long-Term Contaminant Transport.

7.2 Types of Vertical Cutoff Walls

The principal types of vertical cutoff walls are sheet pile walls, geomembrane walls, and slurry trench cutoff walls. Other techniques, such as grouting and deep soil mixing, are also possible, but have rarely been used for waste containment applications.

7.2.1 Sheet Pile Walls

Sheet pile walls are interlocking sections of steel or plastic materials (Fig. 7.5). Steel sheet piles are used for a variety of excavation shoring applications; the same type of steel sheet piles are used for vertical cutoff walls. Plastic sheet piles are a relatively recent development and are used on a limited basis for vertical cutoff walls. Sheet piles measure approximately 0.5 m (18 in.) in width, and interlocks join individual sheets together (Fig. 7.5). Lengths are essentially unlimited, but sheet piles are rarely longer than about 10 to 15 m (30 to 45 ft).

Figure 7.5 - Interlocking Steel Sheet Piles.

Plastic sheet piles are different from geomembrane panels, which are discussed later. Plastic sheet piles tend to be relatively thick-walled (wall thickness > 3 mm or 1/8 in.) and rigid; geomembrane panels tend to have a smaller thickness (< 2.5 mm or 0.1 in.), greater width, and lower rigidity.

Sheet pile walls are installed by driving or vibrating interlocking steel sheet piles into the ground. Alternatively, plastic sheet piles can be used, but special installation devices may be needed, e.g., a steel driving plate to which the plastic sheet piles are attached. To promote a seal, a cord of material that expands when hydrated and attains a very low permeability may be inserted in the interlock. Other schemes have been devised and will continue to be developed for attaining a water-tight seal in the interlock.

Sheet pile walls have a long history of use for dewatering applications, particularly where the sheet pile wall is also used as a structural wall. Sheet pile walls also have been used on several occasions to cutoff horizontal seepage through permeable strata that underlie dams (Sherard et al., 1963).

Sheet pile walls have historically suffered from problems with leakage through interlocks, although much of the older experience may not be applicable to modern sheet piles with expanding material located in the interlock (the expandable material is a relatively recent development).

Leakage through sheet pile interlocks depends primarily on the average width of openings in the interlocking connections, the percentage of the interlocks that leak, and the quality and integrity of any sealant placed in the interlock. The sheet piles may be damaged during installation, which can create ruptures in the sheet pile material or separation of sheet piles at interlocks. Because of these problems, sheet pile cutoffs have not been used for waste containment facilities as extensively as some other types of vertical cutoff walls. Sheet pile walls are not discussed further in this report.

7.2.2 Geomembrane Walls

Geomembrane walls represent a relatively new type of vertical barrier that is rapidly gaining in popularity. The geomembrane wall consists of a series of geomembrane panels joined with special interlocks (examples of interlocks are sketched in Fig. 7.6) or installed as a single unit. If the geomembrane panels contain interlocks, a water-expanding cord is used to seal the interlock.

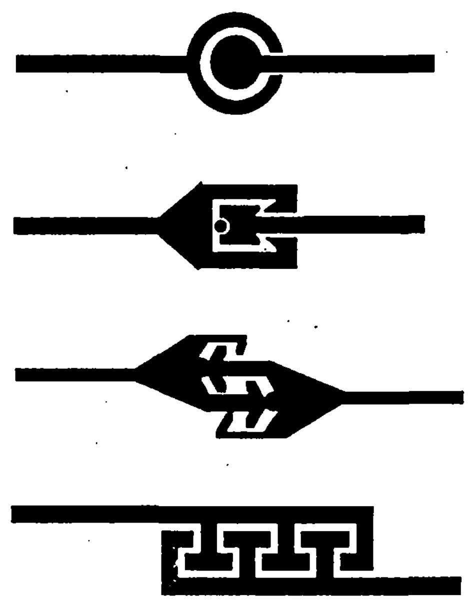

Figure 7.6 - Examples of Interlocks for Geomembrane Walls (Modified from Manassero and Pasqualini, 1992)

The technology has its roots in Europe, where slurry trench cutoff walls that are backfilled with cement-bentonite have been commonly used for several decades. One of the problems with cement-bentonite backfill, as discussed later, is that it is difficult to make the hydraulic conductivity of the cement-bentonite backfill less than or equal to 1×10^{-7} cm/s, which is often required of regulatory agencies in the U.S. To overcome this limitation in hydraulic conductivity and to improve the overall containment provided by the vertical cutoff wall, a geomembrane may be inserted into the cement-bentonite backfill. The geomembrane may actually be installed either in a slurry-filled trench or it may be installed directly into the ground using a special insertion plate.

7.2.3 Walls Constructed with Slurry Techniques

Walls constructed by slurry techniques (sometimes called "slurry trench cutoff walls") are described by Xanthakos (1979), D'Appolonia (1980), EPA (1984), Ryan (1987), and Evans (1993). With this technique, an excavation is made to the desired depth using a backhoe or clamshell. The trench is filled with a clay-water suspension ("mud" or "slurry"), which maintains stability of sidewalls via hydrostatic pressure. As the trench is advanced, the slurry tends to flow into the surrounding soil. Clay particles are filtered out, forming a thin skin of relatively impermeable material along the wall of the trench called a "filter cake." The filter cake has a very low hydraulic conductivity and allows the pressure from the slurry to maintain stable walls on the trench (Fig. 7.7). However, the level of slurry must generally be higher than the surrounding ground water table in order to maintain stability. If the water table is at or above the surface, a dike may be constructed to raise the surface elevation along the alignment of the slurry trench cutoff wall.

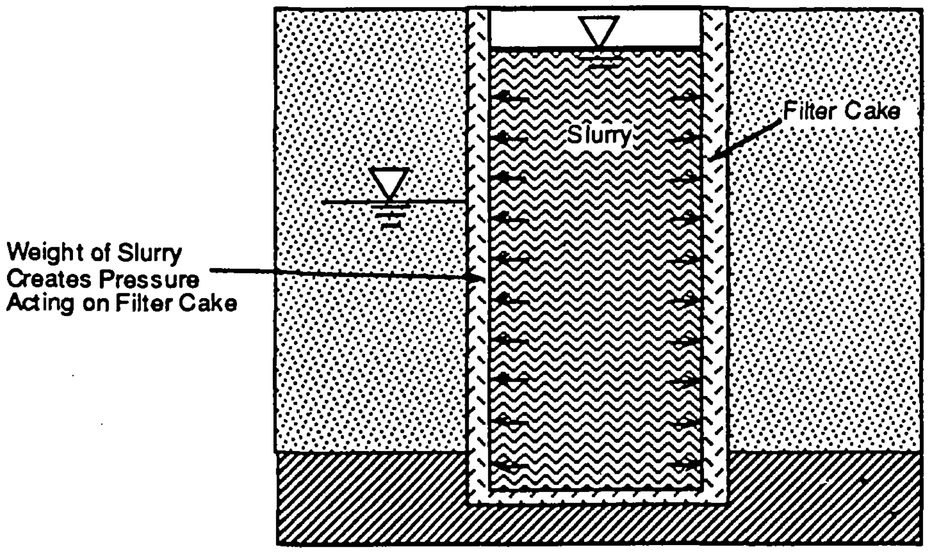

Figure 7.7 - Hydrostatic Pressure from Slurry Maintains Stable Walls of Trench.

In most cases, sodium bentonite is the clay used in the slurry. A problem with bentonite is that it does not gel properly in highly saline water or in some heavily contaminated ground waters. In such cases, an alternative clay mineral such as attapulgite may be used, or other special materials may be used to maintain a viscous slurry.

The slurry trench must either be backfilled or the slurry itself must harden into a stable material -- otherwise clay will settle out of suspension, the slurry will cease to support the walls of the trench, and the walls may eventually collapse. If the slurry is allowed to harden in place, the slurry is usually a cement-bentonite (CB) mixture. If the slurry trench is backfilled, the backfill is usually a soil-bentonite (SB) mixture, although plastic concrete may also be used (Evans, 1993).

In the U.S., slurry trenches backfilled with SB have been the most commonly used vertical cutoff trenches for waste containment applications. In Europe, the CB method of construction has been used more commonly. The reason for the different practices in the U.S. and Europe stems at least in part upon the fact that abundant supplies of high-quality sodium bentonite are readily available in the U.S. but not in Europe. Also, in most situations, SB backfill will have a somewhat lower hydraulic conductivity than cured CB slurry, and in the U.S. regulations have tended to drive the requirements for hydraulic conductivity to lower values than in Europe.

The construction sequence for a soil-bentonite backfilled trench is shown schematically in Fig. 7.8.

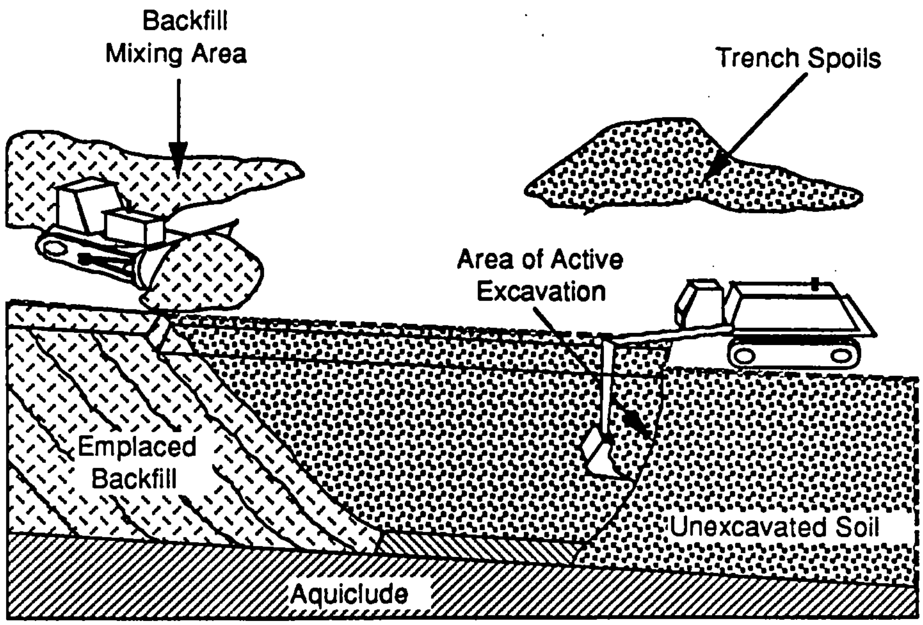

Figure 7.8 - Diagram of Construction Process for Soil-Bentonite-Backfilled Slurry Trench Cutoff Wall.

The main reasons why slurry trench cutoff walls are so commonly used for vertical cutoff walls are:

1. The depth of the trench may be checked to confirm penetration to the desired depth, and excavated materials may be examined to confirm penetration into a particular stratum;

2. The backfill can be checked prior to placement to make sure that its properties are as desired and specified;

3. The wall is relatively thick (compared to a sheet pile wall or a geomembrane wall);

4. There are no joints between panels or construction segments with the most common type of slurry trench cutoff wall construction.

In general, in comparison to sheet-pile walls, deep-soil-mixed walls, and grouted walls, there is more opportunity with a slurry trench cutoff wall to check the condition of the wall and confirm that the wall has been constructed as designed. In contrast, it is much more difficult to confirm that a sheet pile wall has been installed without damage, that grout has fully penetrated all of the desired pore spaces in the soil, or that deep mixing as taken place as desired.

7.3 Construction of Slurry Trench Cutoff Walls

The major construction activities involved in building a slurry cutoff wall are preconstruction planning and mobilization, preparation of the site, slurry mixing and hydration, excavation of soil, backfill preparation, placement of backfill, clean-up of the site, and demobilization. These activities are described briefly in the paragraphs that follow.

7.3.1 Mobilization

The first major construction activity is to make an assessment of the site and to mobilize for construction. The contractor locates the slurry trench cutoff wall in the field with appropriate surveys. The contractor determines the equipment that will be needed, amounts of materials, and facilities that may be required. Plans are made for mobilizing personnel and moving equipment to the site.

A preconstruction meeting between the designer, contractor, and CQA engineer is recommended. In this meeting, materials, construction procedures, procedures for MQA of the bentonite and CQA of all aspects of the project, and corrective actions are discussed (see Chapter 1).

7.3.2 Site Preparation

Construction begins with preparation of the site. Obstacles are removed, necessary relocations of utilities are made, and the surface is prepared. One of the requirements of slurry trench construction is that the level of slurry in the trench be greater than the level of ground water. If the ground water table is high, it may be necessary to construct a dike to ensure that the level of slurry in the trench is above the ground water level (Fig. 7.9). There may be grade restrictions in the construction specifications which will require some regrading of the surface or construction of dikes in low-lying areas. The site preparation work will typically also include preparation of working surfaces for mixing materials. Special techniques may be required for exacavation around utility lines.

7.3.3 Slurry Preparation and Properties

Before excavation begins, as well as during excavation, the slurry must be prepared. The slurry usually consists of a mixture of bentonitic clay with water, but sometimes other clays such as attapulgite are used. If the clay is bentonite, the specifications should stipulate the criteria to be met, e.g., filtrate loss, and the testing technique by which the parameter is to be determined. The criteria can vary considerably from project to project.

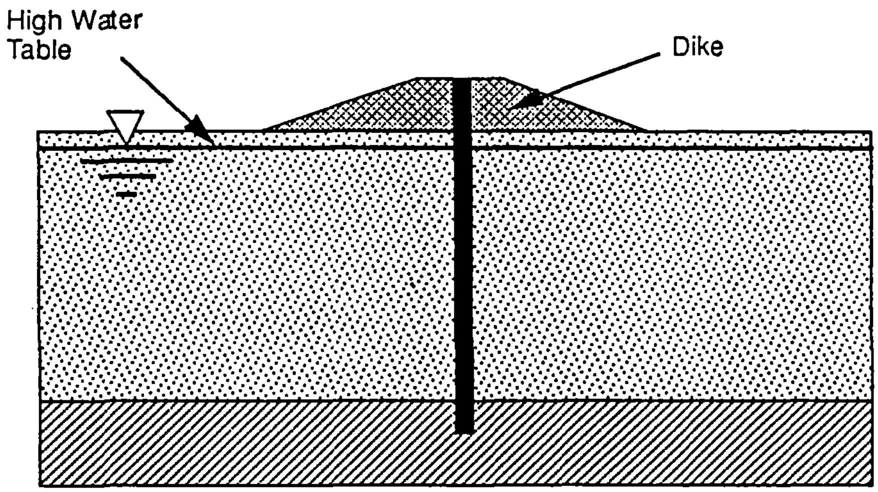

Figure 7.9 - Construction of Dike to Raise Ground Surface for Construction of Slurry Trench.

The clay may be mixed with water in either a batch or flash mixing operation. In the batch system specified quantities of water and bentonite are added in a tank and mixed at high speeds with a pump, paddle mixer, or other device that provides adequate high-speed colloidal shear mixing. Water and clay are mixed until hydration is complete and the desired properties of the slurry have been achieved. Complete mixing is usually achieved in a few minutes. The size of batch mixers varies, but typically a batch mixer will produce several cubic meters of mixed slurry at a time.

Flash mixing is achieved with a venturi mixer. With this system, bentonite is fed at a predetermined rate into a metered water stream that is forced through a nozzle at a constant rate. The slurry is subjected to high shear mixing for only a fraction of a second. The problem with this technique is that complete hydration does not take place in the short period of mixing. After the clay is mixed with water, the resulting slurry is tested to make sure the density and viscosity are within the requirements set forth in the CQA plan.

The mixed slurry may be pumped directly to the trench or to a holding pond or tank. If the slurry is stored in a tank or pond, CQA personnel should check the properties of the slurry periodically to make sure that the properties have not changed due to thixotropic processes or sedimentation of material from the slurry. The specifications for the project should stipulate mixing or circulation requirements for slurry that is stored after mixing.

The properties of the slurry used to maintain the stability of the trench are important. The following pertains to a bentonite slurry that will ultimately be displaced by soil-bentonite or other backfill; requirements for cement-bentonite slurry are discussed later in section 7.3.6. The slurry must be sufficiently dense and viscous to maintain stability of the trench. However, the slurry must not be too dense or viscous: otherwise, it will be difficult to displace the slurry when backfill is placed. Construction specifications normally set limits on the properties of the slurry. Typically about 4-8% bentonite by weight is added to fresh water to form a slurry that has a specific gravity of about 1.05 to 1.15. During excavation of the trench additional fines may become suspended in the slurry, and the specific gravity is likely to be greater than the value of the freshly mixed slurry. The specific gravity of the slurry during excavation is typically on the order of 1.10 - 1.25.

The density of the slurry is measured with the procedures outlined in ASTM D-4380. A known volume of slurry is poured into a special "mud balance," which contains a cup on one end of a balance. The weight is determined and density calculated from the known volume of the cup.

The viscosity of the slurry is usually measured with a Marsh funnel. To determine the Marsh viscosity, fluid is poured into the funnel to a prescribed level. The number of seconds required to discharge 946 mL (1 quart) of slurry into a cup is measured. Water has a Marsh viscosity of about 26 seconds at 23°C. Freshly hydrated bentonite slurry should have a Marsh viscosity in the range of about 40 - 50 seconds. During excavation, the viscosity typically increases to as high as about 65 Marsh seconds. If the viscosity becomes too large the thick slurry must be replaced, treated (e.g., to remove sand), or diluted with additional fresh slurry.

The sand content of a slurry may also be specified. Although sand is not added to fresh slurry, the slurry may pick up sand in the trench during the construction process. The sand content by volume is measured with ASTM D-4381. A special glass measuring tube is used for the test. The slurry is poured onto a No. 200 sieve (0.075 mm openings), which is repeatedly washed until the water running through the sieve is clear. The sand is washed into the special glass measuring tube, and the sand content (volumetric) is read directly from graduation marks.

Other criteria may be established for the slurry. However, filtrate loss and density, coupled with viscosity, are the primary control variables. The specifications should set limits on these parameters as well as specify the test method. Standards of the American Petroleum Institute (1990) are often cited for slurry test methods. Limits may also be set on pH, gel strength, and other parameters, depending on the specific application.

The primarily responsibility for monitoring the properties of the slurry rests with the construction quality control (CQC) team. The properties of the slurry directly affect construction operations but may also impact the final quality of the slurry trench cutoff wall . For example, if the slurry is too dense or viscous, the slurry may not be properly displaced by backfill. On the other hand, if the slurry is too thin and lacks adequate bentonite, the soil-bentonite backfill (formed by mixing soil with the bentonite slurry) may also lack adequate bentonite. The CQA inspectors may periodically perform tests on the slurry, but these tests are usually conducted primarily to verify test results from the CQC team. CQA personnel should be especially watchful to make sure that: (1) the slurry has a sufficiently high viscosity and density (if not, the trench walls may collapse); (2) the level of the slurry is maintained near the top of the trench and above the water table (usually the level must be at least 1 m above the ground water table to maintain a stable trench); and (3) the slurry does not become too viscous or dense (otherwise backfill will not properly displace the slurry).

7.3.4 Excavation of Slurry Trench

The slurry trench is excavated with a backhoe (Fig. 7.10) or a clam shell (Fig. 7.11). Long-stick backhoes can dig to depths of approximately 20 to 25 m (60 to 80 ft). For slurry trenches that can be excavated with a backhoe, the backhoe is almost always the most economical means of excavation. For trenches that are too deep to be excavated with a backhoe, a clam shell is normally used. The trench may be excavated first with a backhoe to the maximum depth of excavation that is achievable with the backhoe and to further depths with a clam shell. Special chopping, chiseling, or other equipment may be used as necessary. The width of the excavation tool is usually equal to the width of the trench and is typically 0.6 to 1.2 m (2 to 4 ft).

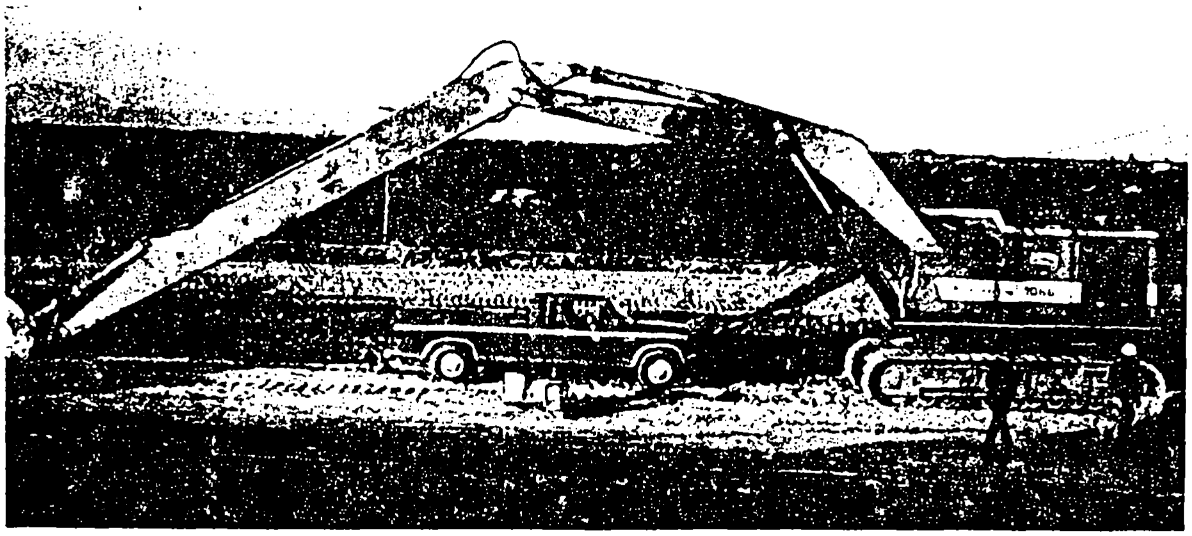

Figure 7.10 - Backhoe for Excavating Slurry Trench.

In most instances, the slurry trench cutoff wall is keyed into a stratum of relatively low hydraulic conductivity. In some instances, the vertical cutoff wall may be relatively shallow. For example, if a floating non-aqueous phase liquid such as gasoline is to be contained, the slurry trench cutoff wall may need to extend only a short distance below the water table surface, depending upon the site-specific circumstances. CQC/CQA personnel monitor the depth of excavation of the slurry trench and should log excavated materials to verify the types of materials present and to ensure specified penetration into a low-permeability layer. Monitoring normally involves examining soils that are excavated and direct measurement of the depth of trench by lowering a weight on a measuring tape down through the slurry. Additional equipment such as air lifts may be needed to remove sandy materials from the bottom of the trench prior to backfill.

7.3.5 Soil-Bentonite (SB) Backfill

Soil is mixed with the bentonite-water slurry to form soil-bentonite (SB) backfill. If the soil is too coarse, additional fines can be added. Dry, powdered bentonite may also be added, although it is difficult to ensure that the dry bentonite is uniformly distributed. In special applications in which the properties of the bentonite are degraded by the ground water, other types of clay may be used, e.g., attapulgite, to form a mineral-soil backfill. If possible, soil excavated from the trench is used for the soil component of SB backfill. However, if excavated soil is excessively contaminated or does not have the proper gradation, excavated soil may be hauled off for treatment and disposal.

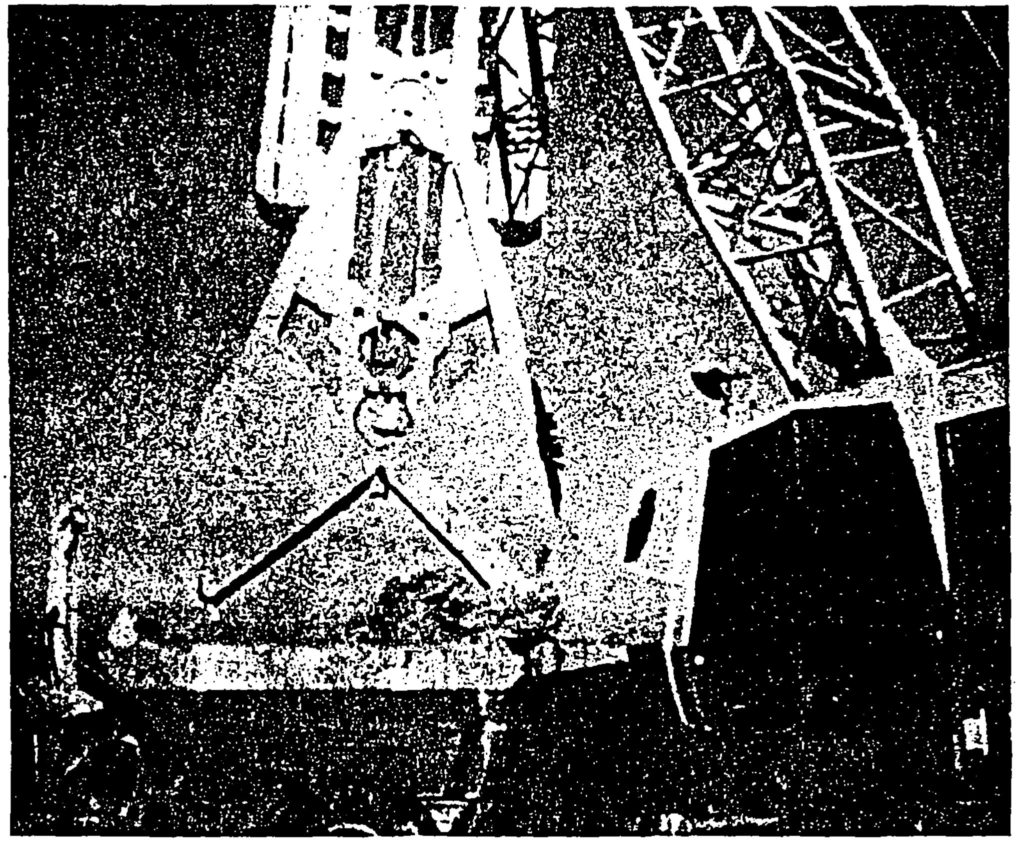

Figure 7.11. Clamshell for Excavating Slurry Trench.

Two parameters concerning the backfill are very important: (1) the presence of extremely coarse material (i.e., coarse gravel and cobbles), and (2) the presence of fine material. Coarse gravel is defined as material with particle sizes between 19 and 75 mm (ASTM D-2487). Cobbles are materials with particle sizes greater than 75 mm. Fine material is material passing the No. 200 sieve, which has openings of 0.075 mm. Cobbles will tend to settle and segregate in the backfill; coarse gravel may also segregate, but the degree of segregation depends on site-specific conditions. In some cases, the backfill may have to be screened to remove pieces that exceed the maximum size allowed in the specifications. The hydraulic conductivity of the backfill is affected by the percentage of fines present (D'Appolonia, 1980; Ryan, 1987; and Evans, 1993). Often, a minimum percentage of fines is specified. Ideally, the backfill material should contain at least 10 to 30% fines to achieve low hydraulic conductivity (< 10^{-7} cm/s).

The bentonite may be added in two ways: (1) soil is mixed with the bentonite slurry (usually with a dozer, as shown in Fig. 7.12) to form a viscous SB material; and (2) additional dry powdered bentonite may be added to the soil-bentonite slurry mixture. Dry, powdered bentonite may or may not be needed. D'Appolonia (1980) and Ryan (1987) discuss many of the details of SB backfill design.

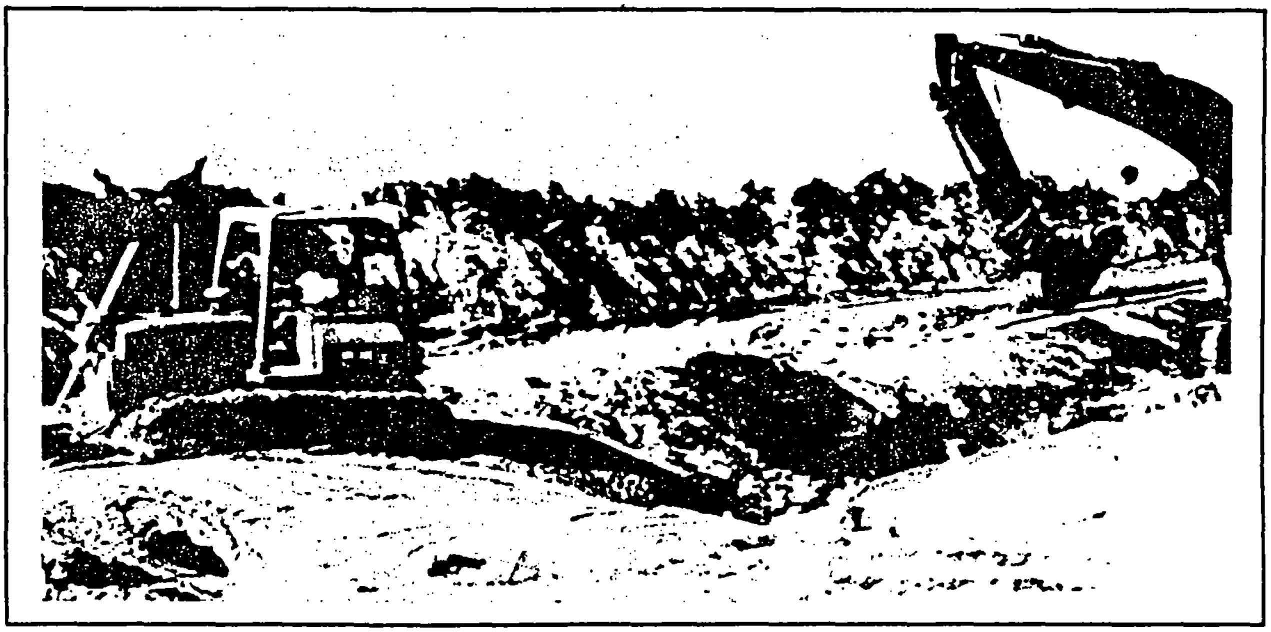

Figure 7.12 - Mixing Backfill with Bentonite Slurry.

When SB backfill is used, a more-or-less continuous process of excavation, preparation of backfill, and backfilling is used. To initiate the process, backfill is placed by lowering it to the bottom of the trench, e.g., with a clamshell bucket, or placing it below the slurry surface with a tremie pipe (similar to a very long funnel) until the backfill rises above the surface of the slurry trench at the starting point of the trench. Additional SB backfill is then typically pushed into the trench with a dozer (Fig. 7.13). The viscous backfill sloughs downward and displaces the slurry in the trench. As an alternative method to initiate backfilling, a separate trench that is not part of the final slurry trench cutoff wall, called a lead-in trench, may be excavated outside at a point outside of the limits of the final slurry trench and backfilled with the process just described, to achieve full backfill at the point of initiation of the desired slurry trench.

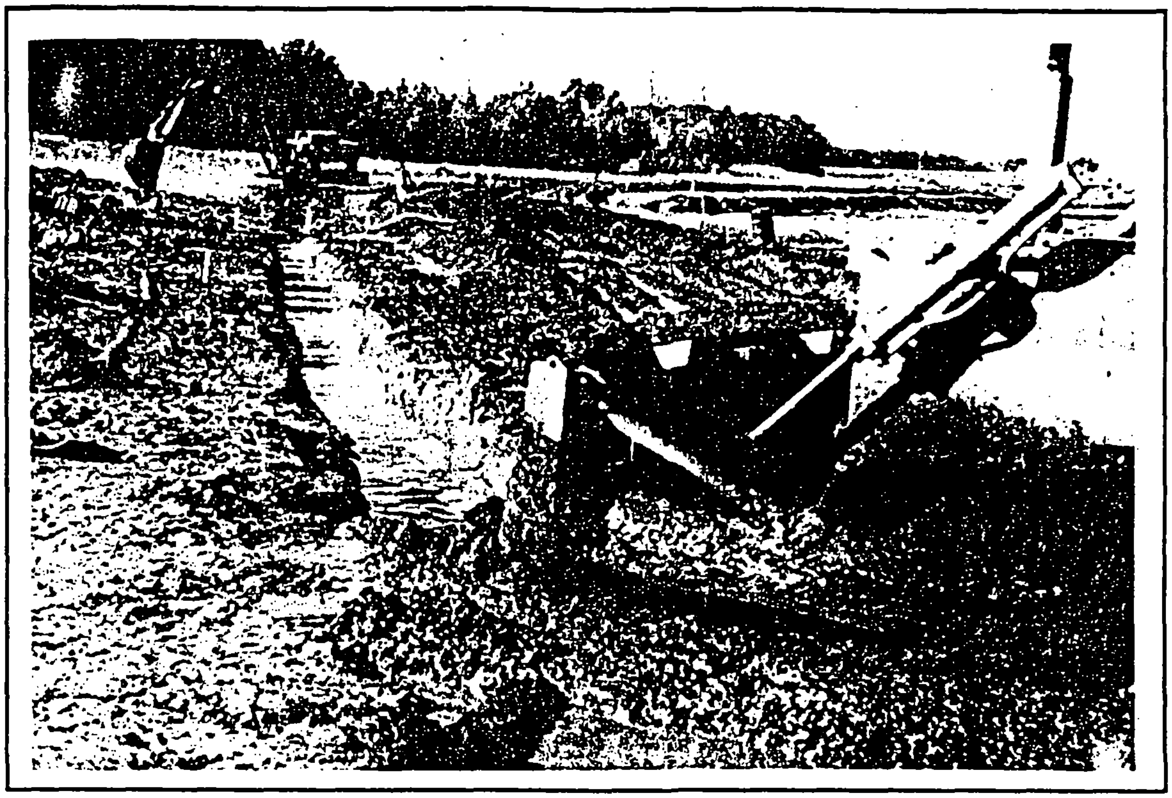

Figure 7.13 - Pushing Soil-Bentonite Backfill Into Slurry Trench with Dozer.

After the trench has been backfilled, low hydraulic conductivity is achieved via two mechanisms: (1) the SB backfill itself has low hydraulic conductivity (typical design value is $\leq 10^{-7}$ cm/s), and (2) the filter cake enhances the overall function of the wall as a barrier. Designers do not normally count on the filter cake as a component of the barrier; it is viewed as a possible source of added impermeability that enhances the reliability of the wall.

The compatibility of the backfill material with the ground water at a site should be assessed prior to construction. However, CQA personnel should be watchful for ground water conditions that may differ from those assumed in the compatibility testing program. CQA personnel should familiarize themselves with the compatibility testing program. Substances that are particularly aggressive to clay backfills include non-water-soluble organic chemicals, high and low pH liquids, and highly saline water. If there is any question about ground water conditions in relationship to the conditions covered in the compatibility testing program, the CQA engineer and/or design engineer should be consulted.

Improper backfilling of slurry trench cutoff walls can produce defects (Fig. 7.14). More details are given by Evans (1993). CQA personnel should watch out for accumulation of sandy materials during pauses in construction, e.g., during shutdowns or overnight; an airlift can be used to remove or resuspend the sand, if necessary.

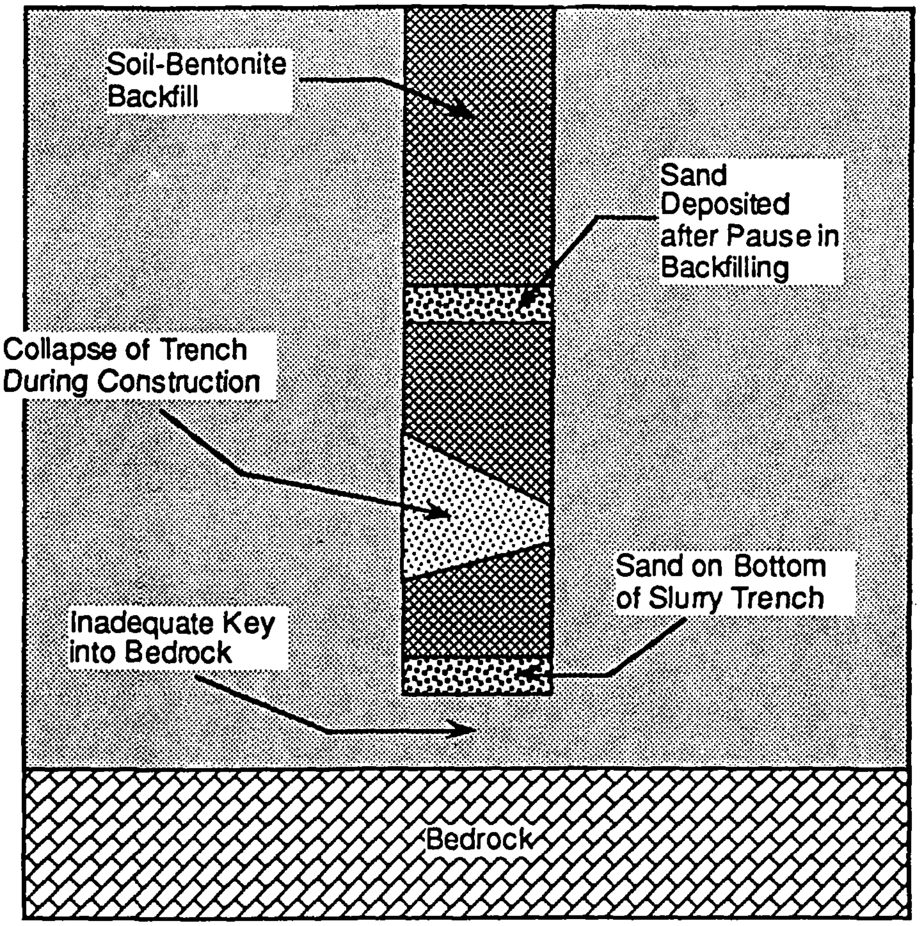

Figure 7.14 - Examples of Problems Produced by Improper Backfilling of Slurry Trench.

Some slurry trench cutoff walls fully encircle an area. As the slurry trench reaches the point of initiation of the slurry trench cutoff wall, closure is accomplished by excavating into the previously-backfilled wall.

Hydraulic conductivity of SB backfill is normally measured by testing of small cylinders of material formed from field samples. Ideally, a sample of backfill material is scooped up from the backfill, placed in a cylinder of a specified type, consolidated to a prescribed effective stress, and permeated. It is rare for borings to be drilled into the backfill to obtain samples for testing.

7.3.6 Cement-Bentonite (CB) Cutoff Walls

A cement-bentonite (CB) cutoff wall is constructed with a cement-bentonite-water mixture that hardens and attains low hydraulic conductivity. The slurry trench is excavated, and excavated soils are hauled away. Then the trench is backfilled in one of two ways. In the usual method, the slurry used to maintain a stable trench during construction is CB rather than just bentonite-water, and the slurry is left in place to harden. A much-less-common technique is to construct the slurry trench with a bentonite-water slurry in discrete diaphragm cells (Fig. 7.15), and to displace the bentonite-water slurry with CB in each cell.

The CB mixture cures with time and hardens to the consistency of a medium to stiff clay (CB backfill is not nearly as strong as structural concrete). A typical CB slurry consists on a weight basis of 75 to 80% water, 15 to 20% cement, 5% bentonite, and a small amount of viscosity reducing material. Unfortunately, CB backfill is usually more permeable than SB backfill. Hydraulic conductivity of CB backfill is often in the range of 10^{-6} to 10^{-5} cm/s, which is about an order of magnitude or more greater than typical SB cutoff walls.

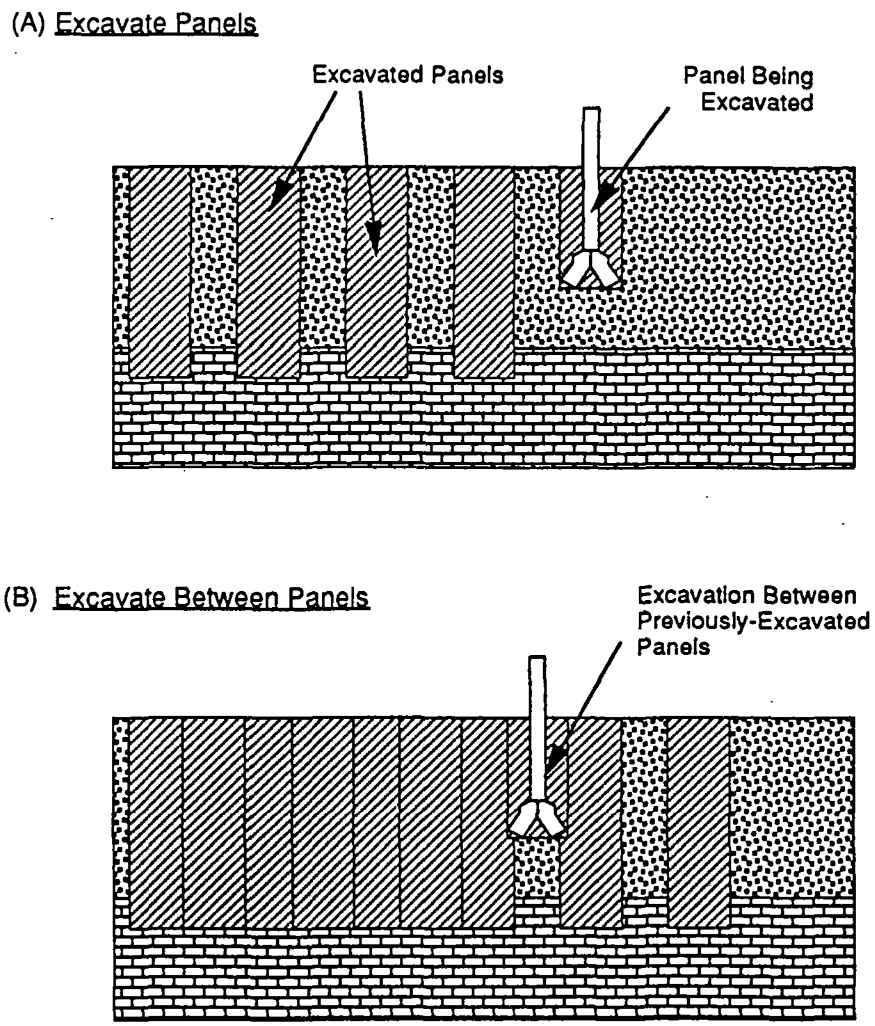

Figure 7.15 - Diaphragm-Wall Construction.

The CB cutoff wall is constructed using procedures almost identical to those employed in building structural diaphragm walls . In Europe, CB backfilled slurry trench cutoff walls are much more common than in the U.S., at least partly because the diaphragm-wall construction capability is more broadly available in Europe and because high-grade sodium bentonite (which is critical for soil-bentonite backfilled walls) is not readily available in Europe. In Europe, the CB often contains other ingredients besides cement, bentonite, and water, e.g., slag and fly ash.

7.3.7 Geomembrane in Slurry Trench Cutoff Walls

Geomembranes may be used to form a vertical cutoff wall. The geomembrane may be installed in one of at least two ways:

1. The geomembrane may be inserted in a trench filled with CB slurry to provide a composite CB-geomembrane barrier (Manassero and Pasqualini, 1992). The geomembrane is typically mounted to a frame, and the frame is lowered into the slurry. The base of the geomembrane contains a weight such that when the geomembrane is released from the frame, the frame can be removed without the geomembrane floating to the top. CQA personnel should be particularly watchful to ensure that the geomembrane is properly weighted and does not float out of position. Interlocks between geomembrane panels (Fig. 7.6) provide a seal between panels. The panels are typically relatively wide (of the order of 3 to 7 m) to minimize the number of interlocks and to speed installation. The width of a panel may be controlled by the width of excavated sections of CB-filled panels (Fig. 7.15).

2. The geomembrane may be driven directly into the CB backfill or into the native ground. Panels of geomembrane with widths of the order of 0.5 to 1 m (18 to 36 in.) are attached to a guide or insertion plate, which is driven or vibrated into the subsurface. If the panels are driven into a CB backfill material, the panels should be driven before the backfill sets up. Interlocks between geomembrane panels (Fig. 7.6) provide a seal between panels. This methodology is essentially the same as that of a sheet pile wall.

Although use of geomembranes in slurry trench cutoff walls is relatiyely new, the technology is gaining popularity. The promise of a practically impermeable vertical barrier, plus excellent chemical resistance of HDPE geomembranes, are compelling advantages. Development of more efficient construction procedures will make this type of cutoff wall increasingly attractive.

7.3.8 Other Backfills

Structural concrete could be used as a backfill, but if concrete is used, the material normally contains bentonite and is termed *plastic concrete* (Evans, 1993). Plastic concrete is a mixture of cement, bentonite, water, and aggregate. Plastic concrete is different from structural concrete because it contains bentonite and is different from SB backfill because plastic concrete contains aggregate. Other ingredients, e.g., fly ash, may be incorporated into the plastic concrete. Construction is typically with the panel method (Fig. 7.15). Hydraulic conductivity of the backfill can be $< 10^{-8}$ cm/s. High cost of plastic concrete limits its use.

A relatively new type of backfill is termed soil-cement-bentonite (SCB). The SCB wall uses native soils (not aggregates, as with plastic concrete). Placement is in a continuous trench rather than panel method.

7.3.9 Caps

A cutoff wall cap represents the final surface cap on top of the slurry trench cutoff wall. The cap may be designed to minimize infiltration, withstand traffic loadings, or serve other purposes. CQA personnel should also inspect the cap as well as the wall itself to ensure that the cap conforms with specification.

7.4 Other Types of Cutoff Walls

Evans (1993) discusses other types of cutoff walls. These include vibrating beam cutoff walls, deep soil mixed walls, and other types of cutoff walls. These are not discussed in detail here because these types of walls have been used much less frequently than the other types.

7.5 Specific CQA Requirements

No standard types of tests or frequencies of testing have evolved in the industry for construction of vertical cutoff walls. Among the reasons for this is the fact that construction materials and technology are continually improving. Recommendations from this section were taken largely from recommendations provided by Evans (personal communication).

For slurry trench cutoff walls, the following comments are applicable. The raw bentonite (or other clay) that is used to make the slurry may have specific requirements that must be met. If so, tests should be performed to verify those properties. There are no standard tests or frequency of tests for the bentonite. The reader may wish to consult Section 2.6.5 for a general discussion of tests and testing frequencies for bentonite-soil liners. For the slurry itself, common tests include viscosity, unit weight, and filtrate loss, and other tests often include pH and sand content. The properties of the slurry are normally measured on a regular basis by the contractor's CQC personnel; CQA personnel may perform occasional independent checks.

The soil that is excavated from the trench should be continuously logged by CQA personnel to verify that subsurface conditions are similar to those that were anticipated. The CQA personnel should look for evidence of instability in the walls of the trench (e.g., sloughing at the surface next to the trench or development of tension cracks). If the trench is to extend into a particular stratum (e.g., an aquitard), CQA personnel should verify that adequate penetration has occurred. The recommended procedure is to measure the depth of the trench once the excavator has encountered the aquitard and to measure the depth again, after adequate penetration is thought to have been made into the aquitard.

After the slurry has been prepared, and CQC tests indicate that the properties are adequate, additional samples are often taken of the slurry from the trench. The samples are often taken from near the base of the trench using a special sampler that is capable of trapping slurry from the bottom of the trench. The unit weight is particularly important because sediment may collect near the bottom of the trench. For SB backfill, the slurry must not be heavier than the backfill. The depth of the trench should also be confirmed by CQA personnel just prior to backfilling. Often, sediments can accumulate near the base of the trench -- the best time to check for accumulation is just prior to backfilling. CQA personnel should be particularly careful to check for sedimentation after periods when the slurry has not been agitated, e.g., after an overnight work stoppage.

Testing of SB backfill usually includes unit weight, slump, gradation, and hydraulic conductivity. Bentonite content may also be measured, e.g., using the methylene blue test (Alther, 1983). Slump testing is the same as for concrete (ASTM C-143). Hydraulic conductivity testing is often performed using the API (1990) fixed-ring device for the filter press test. Occasional

..mparative tests with ASTM D-5084 should be conducted. There is no widely-applied frequency of testing backfill materials.

7.6 Post Construction Tests for Continuity

At the present time, no testing procedures are available to determine the continuity of a completed vertical cutoff wall.

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